Appendix D

Numerical Model Assessment of Bed Shear Stress for Wind-Waves and Flows on LLBdM (OU1), Fox River (Baird 2007)

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Numerical Model Assessment of Bed Shear Stress for Wind-Waves and Flows on Little Lake Butte des Morts (OU1), Fox River

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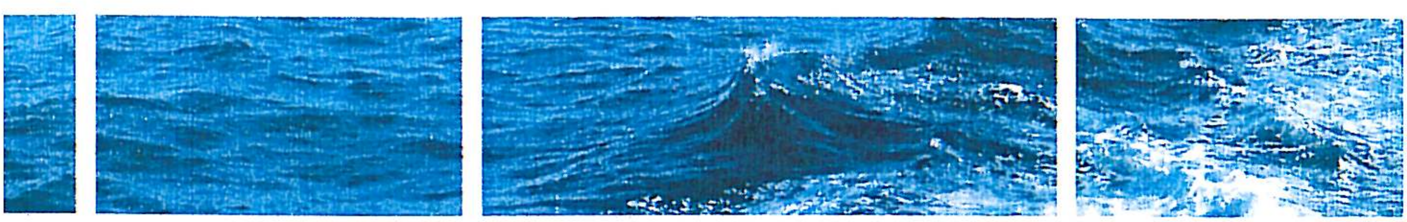

Navigating New Horizons

Numerical Model Assessment of Bed Shear Stress for Wind-Waves and Flows on Little Lake Butte des Morts (OU1), Fox River

Prepared for

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1.0 INTRODUCTION

Baird was retained by Foth Infrastructure & Environment, LLC (Foth) to evaluate bed shear stresses generated by wind-waves and flows in Little Lake Butte des Morts. Little Lake Butte des Morts occupies the upper part of the Lower Fox River from Lake Winnebago to the Appleton Dam, and is referred to as Operable Unit 1 (OU1) of the Lower Fox River Project. The evaluation was performed on a projected post-remedy OU1 bathymetric condition; proposed OU1 optimized Remedy (13 and 16 inch cap placeholders were used in the analysis). The work follows from the project to estimate shear stresses for OU1 under the 100-year flow (with no wind-waves) described in Baird (2006) and based on the ECOMSED model of the Lower Fox River developed and tested by Baird (Baird, 2000). The model results provide information that will be used, in conjunction with other studies, to evaluate the overall stability of the cap materials for a proposed sediment remediation project in Little Lake Butte des Morts.

2.0 BACKGROUND AND APPROACH

2.1 Objectives

The stated objectives of this assignment were as follows:

Determine the approximate combined shear stresses in Little Lake Butte des Morts under various wave and current conditions. Model outcomes will be considered within the design process of a proposed cap for OU1. The river is always flowing through Little Butte des Morts, therefore, shear stresses created by wind-waves must be considered in combination with river flows. Also, a review of liquefaction and wave pumping issues, specific to OU1, has been completed.

2.2 Selected Wind-Wave and Flow Combinations

It is noted that Foth provided direction on the selection of five conditions required to provide information for Foth to address shear stress in the cap design. The five conditions consisted of:

Run 1 - 2-year return period river flow with 50-year return period wind from SSW

Run 2 - 2-year return period river flow with 50-year return period wind from WSW

Run 3 - 2-year return period river flow with 50-year return period wind from NNW

Run 4 - Average daily flow with a 9-year return period wind from the SSW

Run 5 - Average daily flow with a 2.5-year return period wind from the NNE

Runs 1 to 3 represent three possible combinations of a combined 100-year event or return period on shear stress conditions. Runs 4 and 5 settings were used to simulate more common wind-wave conditions, for purposes of comparison with related studies on OU2-5 (Shaw and Anchor, 2006). Return periods for winds are based on hourly wind data and return periods for flows are based on daily data.

An analysis was completed to evaluate whether there was any interdependency between high flows and high winds speeds. It was determined that these two phenomena were relatively independent. Therefore, it is appropriate to multiply the return periods for the wind and flow to determine the approximate return period of the combined event. Had there been statistical dependency between high winds and flows, the 100-year event would have consisted of larger combined events. It is noted, however, that there are many other possible 100-year event combinations (such as the 100-year wind with a 1.1-year flow condition, the 100-year flow with a 1.1-year wind event, etc.). Some of these combinations may result in higher combined shear stresses than the events selected in Runs 1 to 3 above. The wind speed analysis to review possible interdependency and to determine return period is presented in Section 2.3.

Runs 4 and 5 were used to determine wind-wave outcomes for conditions that were similar to those used for the wind-wave analysis conducted for OU2 to OU5 (Shaw and Anchor, 2006). These runs differed from Runs 1 to 3 in the following ways:

- The return periods for winds were determined assuming no ice cover (in other words, the winds for the entire year were used);
- The return periods for winds were selected to be similar to model runs from the OU2-5 wind-wave analysis (Shaw and Anchor, 2006), for the SSW and NNE wind directions.
- The resulting shear stress combination of wind-waves and flows was not meant to represent a 100-year event;

With respect to water level, Runs 1 to 3 were completed with a water level of 0.2 m (measured at the Appleton Dam and the downstream end of Little Lake Buttes des Morts) above LWD which is representative of a 2-year return period flow condition. The water level for Runs 4 and 5 with the average annual flow were completed with a water level of 0.1 m below LWD, representing a very low water condition.

2.3 Analysis of Wind Data

Wind data was analyzed for two different nearby airports, Appleton-Outagamie County Regional Airport and Green Bay-Austin Straubel International Airport. The Appleton record covered the period from January 1998 to December 2002. A Green Bay record from 1978 to 2002 was obtained to provide a longer dataset to support a more accurate extreme values analysis. A portion of the Green Bay data was recorded at an anemometer elevation of 20 ft (6.1 m), and it was assumed that the balance of the data (for which no elevation was provided) was also recorded at that elevation. The elevation at which the Appleton data was measured but was not reported by the weather station, so it was assumed to be at the standard height of 32.8 ft (10 m) based on direction from Foth. The wind data itself was separated into two separate seasons: full-year and open-water (when ice is not present). An analysis of 25 years of ice data on the southern tip of Green Bay was used in conjunction with an OU1 project ice study (Ashton, 1996) to determine an approximate average ice season; the analysis and data review resulted in the approximate annual average open water season being defined as April 1st through December 20th.

An analysis was undertaken to determine the dependency between river flows and wind speeds and a consistent relationship or dependency between the two datasets was not observed.

An extreme value analysis was undertaken on both datasets, and for both seasons. There was insufficient data at Appleton to determine return periods beyond 10 years with a reasonable level of certainty. The Peaks Over Threshold (POT) approach was implemented with individual events selected on the basis of storm intensity, duration, and direction. Results for both seasons are shown in Tables 1 and 2 (note: 1 mph = 0.447 m/s). Four separate statistical distributions were used to represent the tail of the data in each case, and the final distribution was selected on the basis of goodness of fit measures and engineering judgment. More complete information on the POT analysis for each direction and season is provided in Appendix B.

Table 1
Return Period Events for Various Wind Speeds: Full Year

Return Period	Green Bay Wind Speeds (mph)				Appleton Wind Speeds (mph)			
(years)	SSW	SW	NNE	NE	SSW	SW	NNE	NE
1	29.3	31.0	28.2	29.6	29.7	33.8	22.0	30.1
2	31.7	34.3	30.4	31.0	34.1	36.5	29.0	31.4
5	35.1	38.7	33.5	33.1	40.3	40.0	33.3	33.0
10	38.3	42.0	35.7	34.8	45.6	42.5	36.1	34.2
50	48.9	49.8	41.0	39.0	N/A	N/A	N/A	N/A
100	55.5	53.1	43.3	40.8	N/A	N/A	N/A	N/A

Table 2
Return Period Events for Various Wind Speeds: Open-Water Season

Return Period	Green Bay Wind Speeds (mph)				Appleton Wind Speeds (mph)			
(years)	SSW	SW	NNE	NE	SSW	SW	NNE	NE
1	26.7	27.8	21.7	27.3	29.2	29.2	N/A	28.2
2	30.6	31.1	26.3	29.2	34.1	31.6	N/A	28.8
5	34.8	34.7	29.1	31.3	40.6	35.3	N/A	30.6
10	38.4	37.8	31.0	33.2	45.4	38.6	N/A	32.6
50	49.1	47.1	35.0	38.6	N/A	N/A	N/A	N/A
100	55.1	52.3	36.6	41.7	N/A	N/A	N/A	N/A

Runs 1, 2, and 3 were conducted using winds speeds with a 50-year return period. The wind speeds for the SSW, WSW, and NNW conditions for Runs 1, 2, and 3 are shown in Table 3. Runs 4 and 5 were to use wind speeds for return periods that were found to match two of the modeled OU2-5 conditions (Shaw and Anchor, 1996). The return period of the OU2-5 events was determined for their input winds and correlated to Appleton winds. Run 4 was selected to match the OU2-5 condition of a 37.3 mph wind from the SSW: this corresponded to an 8.2-year return period event with the Green Bay winds, which converted to a 44 mph wind (from the SSW) at Appleton. Run 5 was selected to match the OU2-5 condition of a 31.1 mph wind from the NNE: this corresponded to a 2.5-year return period event with the Green Bay winds, which converted to a 30.2 mph wind (from the SSW) at Appleton.

2.4 Numerical Modeling of Wave Conditions

Wind wave conditions in Little Lake Butte des Morts (Fox River OU1) were simulated using the STWAVE model developed by the US Army Corps of Engineers (USACE, 2001). STWAVE is a phase-averaged, steady-state, half plane two dimensional spectral wave model based on the wave action balance equation, which includes wind generation, refraction, shoaling, breaking, limited implementation of wave diffraction, wave-wave interaction and white-capping that redistribute and dissipate energy in a growing wave field. For wave generation, the steady-state assumption means that the winds have remained steady sufficiently long that waves are not limited by the duration of the winds, a reasonable assumption for the fetch-limited conditions in the Little Lake Butte des Morts. The water level used for the lake surface during the STWAVE model runs was +0.2 m (~7.9 inches), related to low water datum.

The model is run on a regularly spaced grid. Different grids were prepared for the model, to properly simulate wind conditions from SSW, WSW, NNW and NNE; the X-Axis of the model grid is oriented parallel to the desired wind direction. 10m-grid resolution was used for all the grids. The model grids were prepared using the post remedy bathymetry (proposed OU1 plan using 13 and 16 inch, cap placeholder designs) related to low water datum. Wisconsin State Plane South in feet (provided to Baird by Foth in April 2007). Horizontal coordinate system was re-projected to match the horizontal coordinate system of earlier model runs (WTM). Figure 1 shows the bathymetry grid used for SSW.

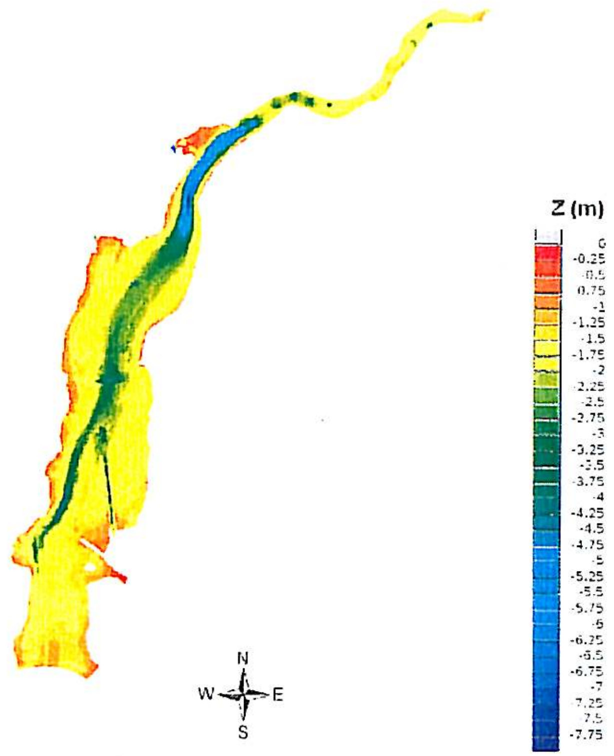

Figure 1. Model Bathymetry for SSW Grid

Five different wind conditions were simulated using the respective bathymetry grids. The wind conditions are shown in Table 3 below. Output from the model consists of wave height, period and direction at each point in the model grid, providing a map of these parameters throughout the model domain.

Table 3
Wind Speed and Direction with Combined Flow Condition for STWAVE Simulations

Run	Wind Direction	Wind Speed (mph)	Wind Speed (m/s)	Return Period (wind, yrs)	Flow Condition (cms)	Flow Description
1	SSW	52.8	23.6	50	360	2 Yr Return Flow
2	WSW	50.6	22.6	50	360	2 Yr Return Flow
3	NNW	37.6	16.8	50	360	2 Yr Return Flow
4	SSW	44.1	19.7	8.2	122	Mean Flow
5	NNE	30.2	13.5	8.2	122	Mean Flow

The predicted wave heights and periods for Run 1 (SSW wind at 52.8 mph) are shown in Figures 2a and 2b below. The wave height and period maps for the other runs are presented in Appendix A.

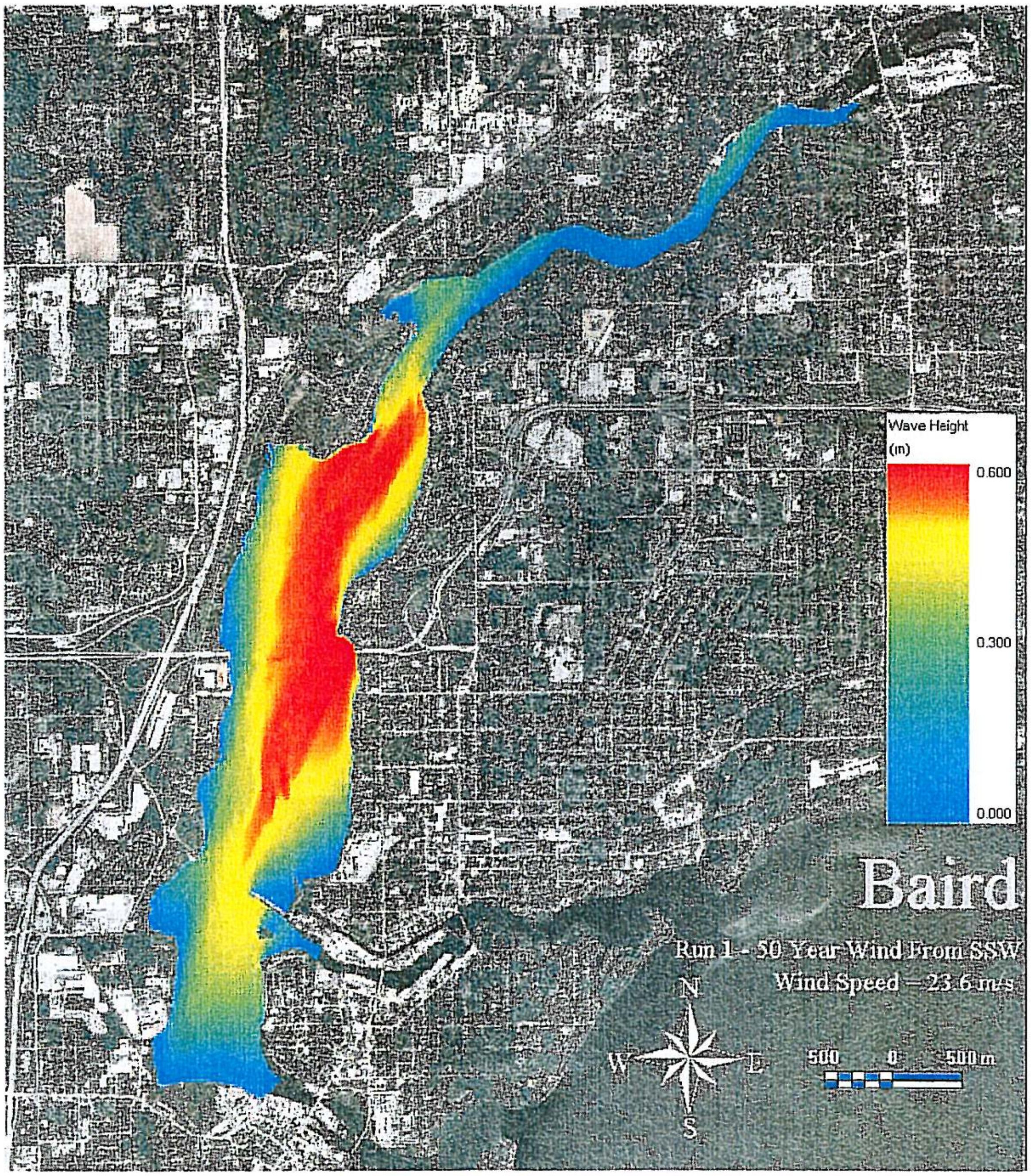

Figure 2a. Significant Wave Heights Predicted with STWAVE for a SSW Wind at 52.8 mph

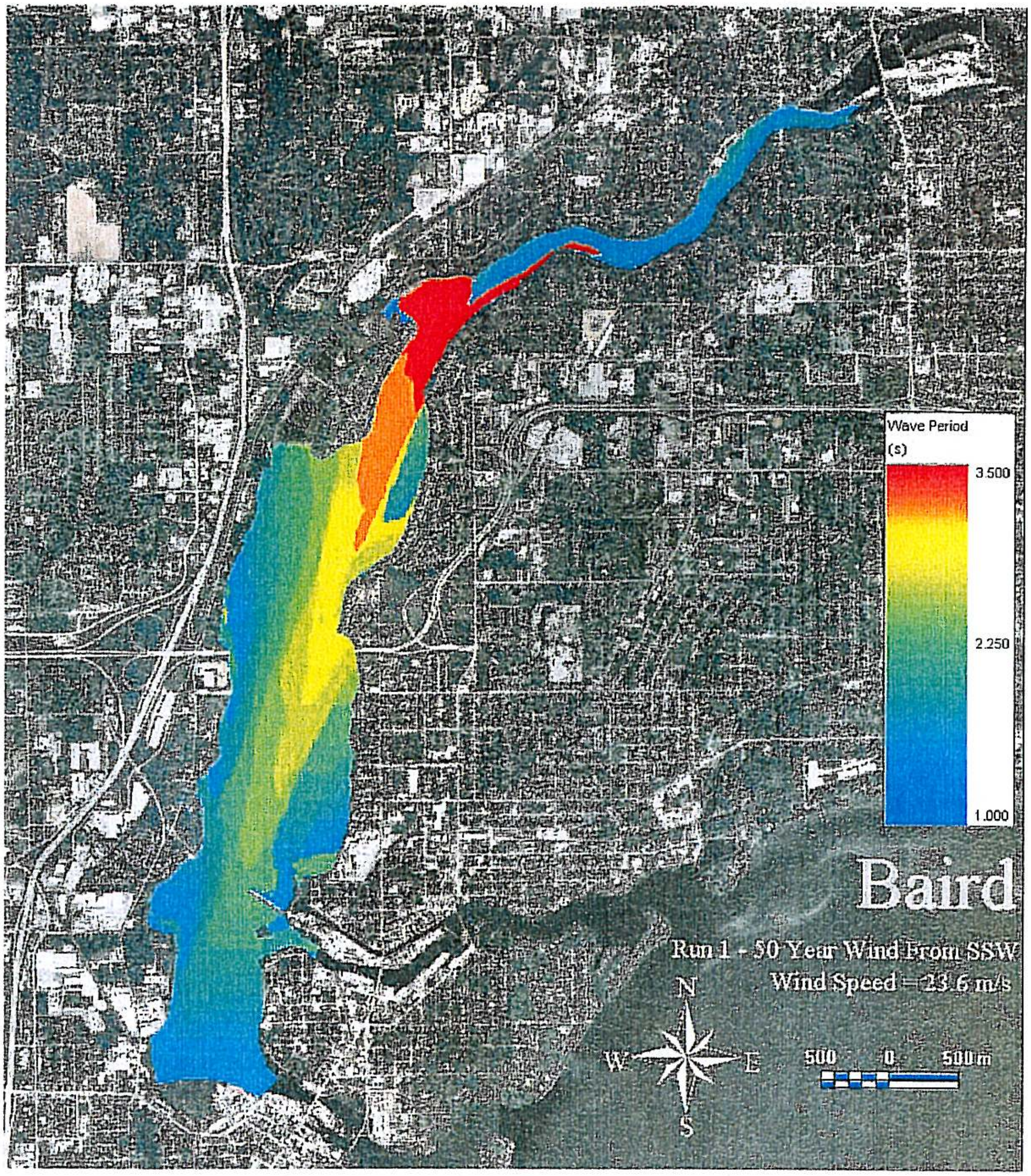

Figure 2b. Wave Periods Predicted with STWAVE for a SSW Wind at 52.8 mph

2.5 Numerical Modeling of River Flow Conditions

A discussion of revisions to the ECOMSED model for the estimates of flow speeds and directions are provided in Baird (2006). The model bathymetry was revised again for this investigation based on information provided by Foth related to revisions to the cap design since the writing of the Baird (2006) report in October 2006. For this study, the daily average (4,300 cfs or 122 m^3/s) and 2-year return period (12,710 cfs or 360 m^3/s) flows were simulated with ECOMSED model for use with Runs 4/5 and Runs 1-3, respectively (i.e. to add to the 100-year return period flow results from Baird, 2006). The flow speeds for two events are shown in Figures 3a and 3b.

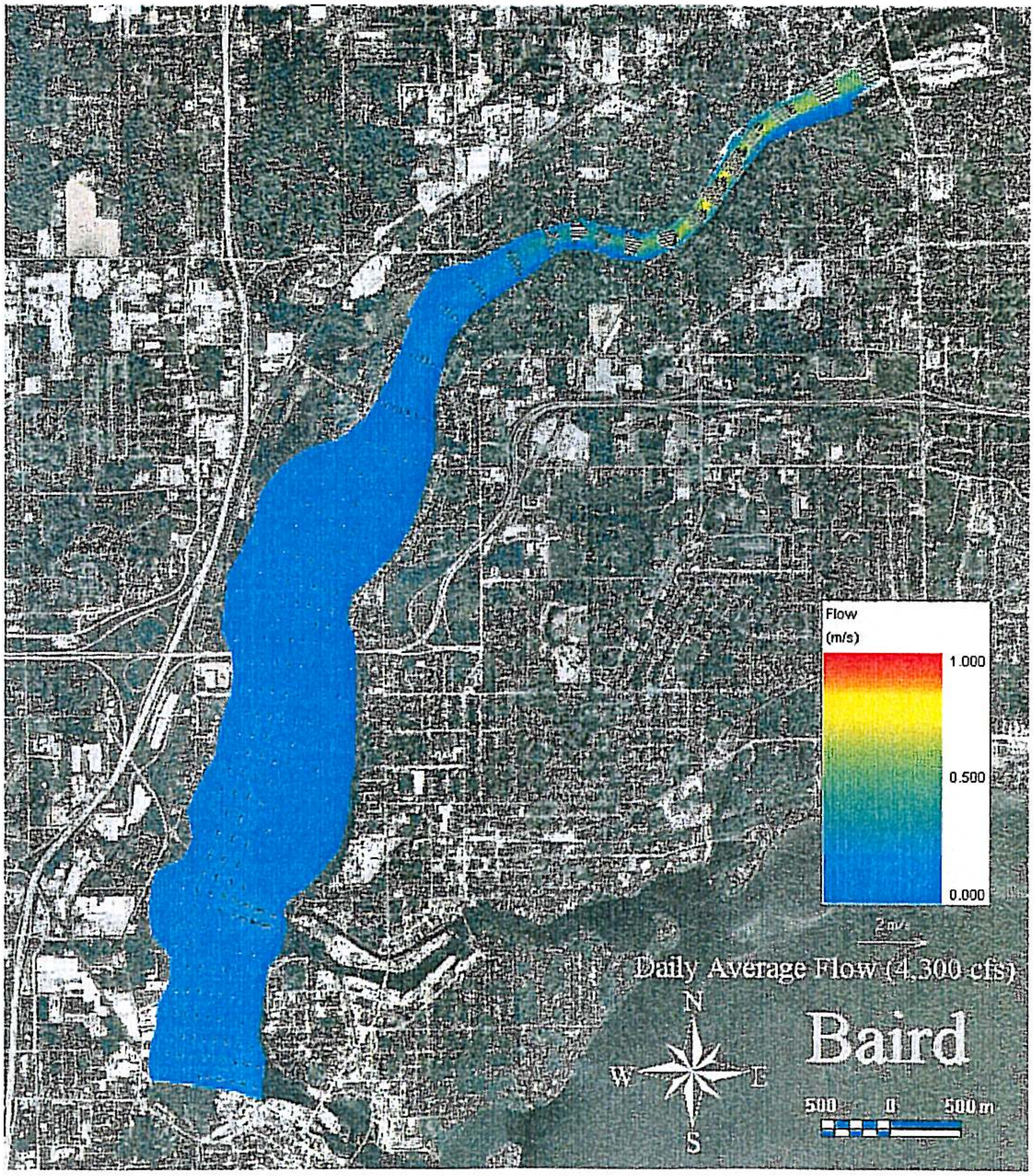

Figure 3a. Flow Speeds Predicted with ECOMSED for the Daily Average Condition

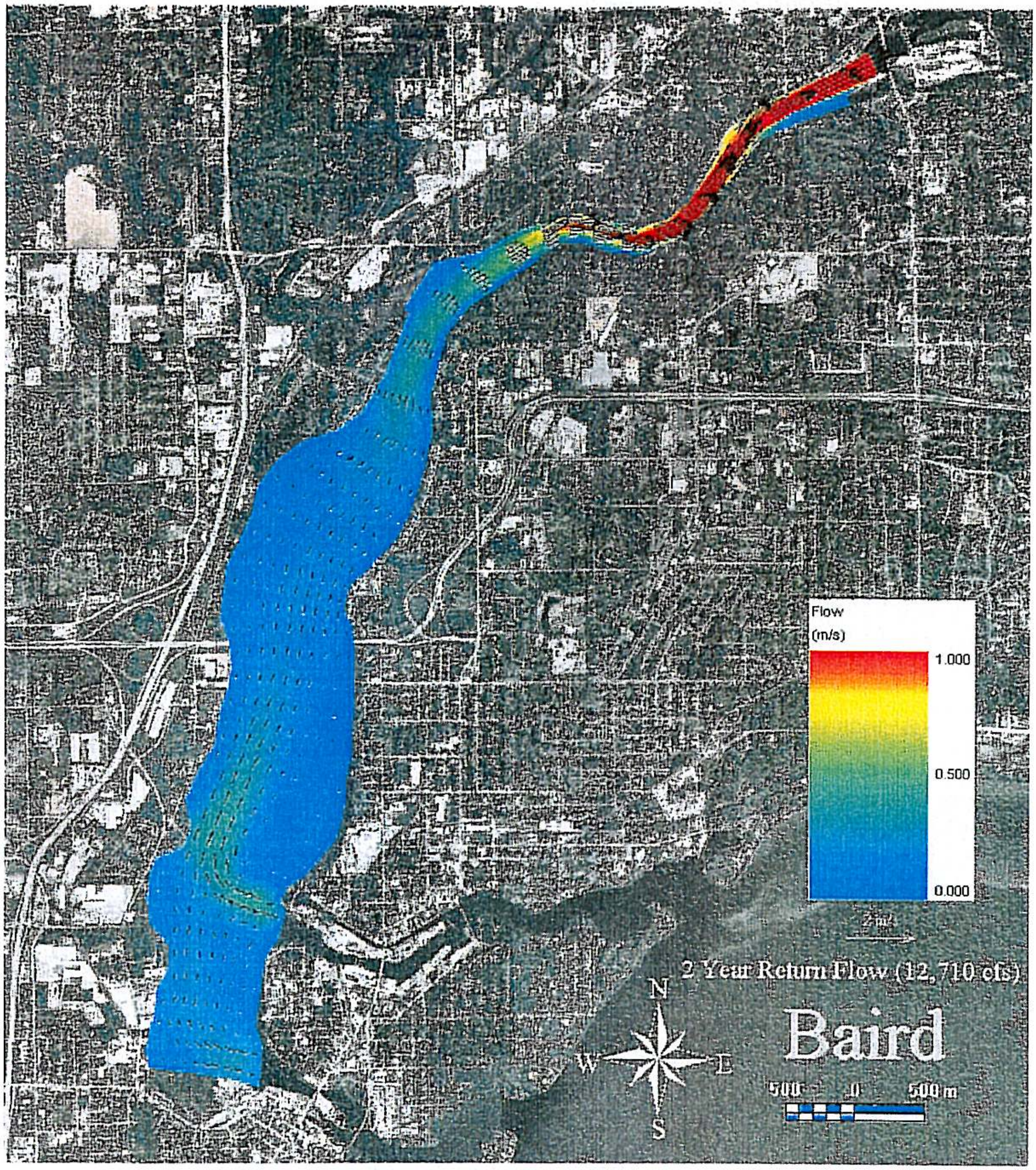

Figure 3b. Flow Speeds Predicted with ECOMSED for the 2-year Return Period Condition

2.6 The Role of Wind-Generated Currents

The section of the Lower Fox River between Lake Winnebago and the Appleton Dam is effectively a lake. Therefore, in addition to the orbital velocities generated by wind-waves and the river flow, there will also be circulation generated by wind shear stresses on the surface of the lake. The wind-driven circulation pattern was considered for Run 1 only to evaluate the importance of this component of the flow on the lake. The results are presented in Section 3.2.

2.7 Methodology for Determining Shear Stresses

Three sets of shear stress maps were developed for each of the model runs: wave-generated shear stresses; river flow generated shear stresses; and combined wave-current generated shear stresses. All three shear stress maps were generated at the 10 m by 10 m grid resolution of the STWAVE model by linear interpolation of the coarser and curvilinear grid of the ECOMSED model (the latter is described in Baird, 2006).

The bed shear stress generated by waves and current was calculated by using the van Rijn (1993) equation:

$$\tau_{b,cw} = \alpha \cdot \tau_{b,c} + \tau_{b,w}$$

in which $\tau_{b,cw}$ is the bed shear stress produced by the combination of waves and currents, $\tau_{b,c}$ is the bed shear stress produced by currents only, $\tau_{b,w}$ is the bed shear stress produced by waves only, and the coefficient α is the bed shear stress reduction factor (<= 1) to account for the current reduction at the bottom due to the present waves. The equation for the coefficient α is

$$\alpha = \left[\frac{\ln(30\delta / k_a)}{\ln(30\delta / k_s)}\right]^2 \left[\frac{-1+\ln(30h/k_s)}{-1+\ln(30h/k_a)}\right]^2$$

in which:

δ - thickness of wave-related near bed layer (=$3\delta_w$ and < $0.033k_a$) (m),

δ_ω - thickness of wave boundary layer ($= 0.072\hat{A}_\delta \left(\frac{\hat{A}_\delta}{k_s}\right)^{-0.25}$)(m)

h - water depth (m)

k_s - bed roughness (m)

k_a - apparent bed roughness (m)

$\hat{A}_\delta$ - wave orbital excursion ($= \frac{H}{2\sinh(kh)}$) (m)

H - wave height (m)

k - wave number ($= 2\pi / L$) (1/m)

L - wave length ($= (gT^2 / 2\pi)\tanh(kh)$) (m)

T - wave period (s)

The bed shear stress produced by current only was calculated by:

$$\tau_{b,c} = \rho g \frac{\overline{u}}{C^2}$$

in which:

ρ - fluid density (kg/m^3)
g – acceleration of gravity (m/s^2)
$\overline{u}$ - depth-averaged velocity (m/s)
C – Chezy coefficient (m$^{0.5}$/s)

$$C = \begin{cases} 18\log\left(\dfrac{11.4h}{\nu C/\overline{u}}\right) & \textit{for hydraulic smooth flow} \\ 18\log\left(\dfrac{12h}{k_s + 1.05\nu C}/\overline{u}\right) & \textit{for transitional flow} \\ 18\log\left(\dfrac{12h}{k_s}\right) & \textit{for hydraulic rough flow} \end{cases}$$

ν - kinematic viscosity coefficient

The bed shear stress produced by waves only was calculated by:

$$\tau_{b,w} = 0.5\rho f_w \hat{U}_\delta^2$$

in which:

$\tau_{b,w}$ – instantaneous wave-related bed shear stress (N/m^2)
f_w – friction coefficient (-)

$$= \begin{cases} 2\left(\dfrac{\hat{U}_\delta \hat{A}_\delta}{\nu}\right)^{-0.5} & \text{for Laminar flow} \\ 0.09\left(\dfrac{\hat{U}_\delta \hat{A}_\delta}{\nu}\right)^{-0.2} & \text{for smooth turbulent flow} \\ \exp\left[-6 + 5.2\left(\dfrac{\hat{A}_\delta}{k_s}\right)^{-0.19}\right] & \text{for rough turbulent flow } (f_{w,\max} = 0.3 \text{ for } \dfrac{\hat{A}_\delta}{k_s} \le 1.57) \end{cases}$$

$\hat{U}_\delta$ - peak orbital velocity just outside boundary layer (m/s) = $\dfrac{\pi H}{T\sinh(kh)}$

3.0 RESULTS ON ESTIMATED SHEAR STRESSES FOR OU1, FOX RIVER

As explained in Section 2.7, three shear stress maps were produced for each of the five model runs corresponding to: shear stresses from river flow alone; shear stresses from wind-wave generated orbital velocities; and combined wave-current generated shear stresses. It is not appropriate to simply superimpose the wave and flow generated shear stresses due to the wave-current interaction in the boundary layer.

In all of the shear stress bed maps the color mapping scheme is the same as that used in Baird (2006), with the exception that the results are presented in Pa and not dynes/cm^2. The color-mapping scheme is described in Table 4 below and it provides an indication of the range of cap sizes required in different areas.

Table 4
Color Mapping Scheme in the Shear Stress Plots

Cap Materials	Grain Size	Inches	Critical Shear Stress for Erosion (Pa)	(Dynes/cm^2)	Color
Medium Sand	< 0.5 mm	< 0.020	0.25	2.5	Navy Blue
Very Coarse Sand	< 2 mm	< 0.079	1.3	13	Greenish Yellow
Fine Gravel	< 8 mm	< 0.315	7.1	71	Green/Blue/Teal
Median Gravel	< 16 mm	< 0.630	14.2	142	Pink/Red

Note: 10 dynes/cm^2 = 1 Pa

The three shear stress maps for each of the five runs have also been provided as GIS files as part of the deliverable for this project (see Section 7.0).

3.1 Shear Stresses Generated by River Flows

Two river flows were considered for the five model runs: the 2-year return period flow (Runs 1 to 3) and the average daily flow (Runs 4 and 5). The river flows were selected to simulate close to a 100-year combined event, with greater impact being placed on the wind event. An analysis into the dependency between the flow and wind datasets was completed and there was almost no dependency observed; therefore combining the 2-year flow and the 50-year wind resulted in an approximate 100-year combined event. Later, as will be explained, it was discovered that lower flow events result in lower water levels on Little Lake Buttes des Morts and therefore, higher wave driven shear stresses due to shallower water depths. Figures 4a and 4b show the shear stress map results for the 2-year return flow and average daily flow, respectively. The shear stresses for river flows alone are low for both these cases and almost everywhere with the cap zone they are less than 1 Pa or 10 dynes/cm^2 for the 2-year event and less than 0.25 Pa for the average daily flow event.

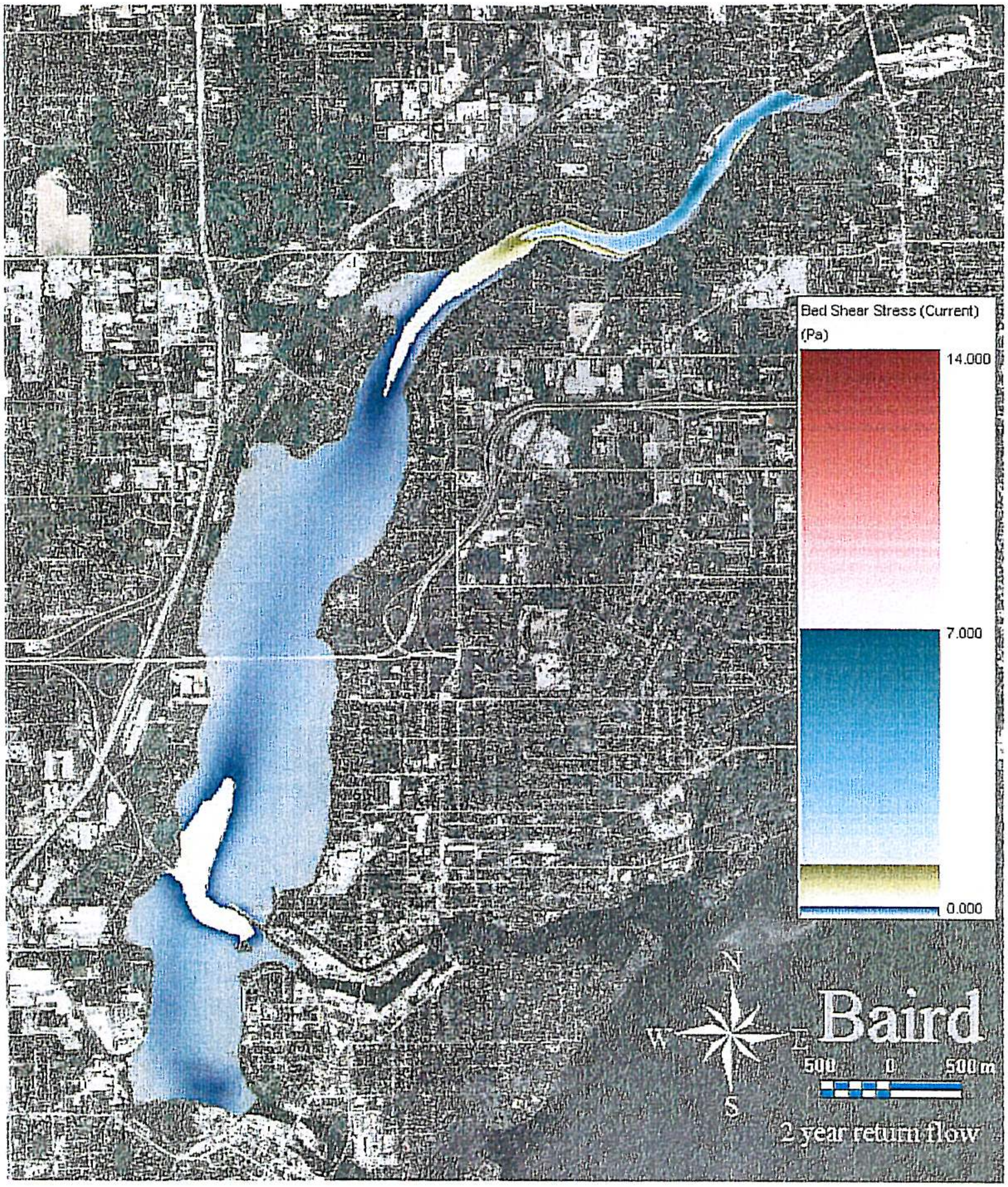

Figure 4a. Runs 1 to 3 – River Currents Only Shear Stress Map

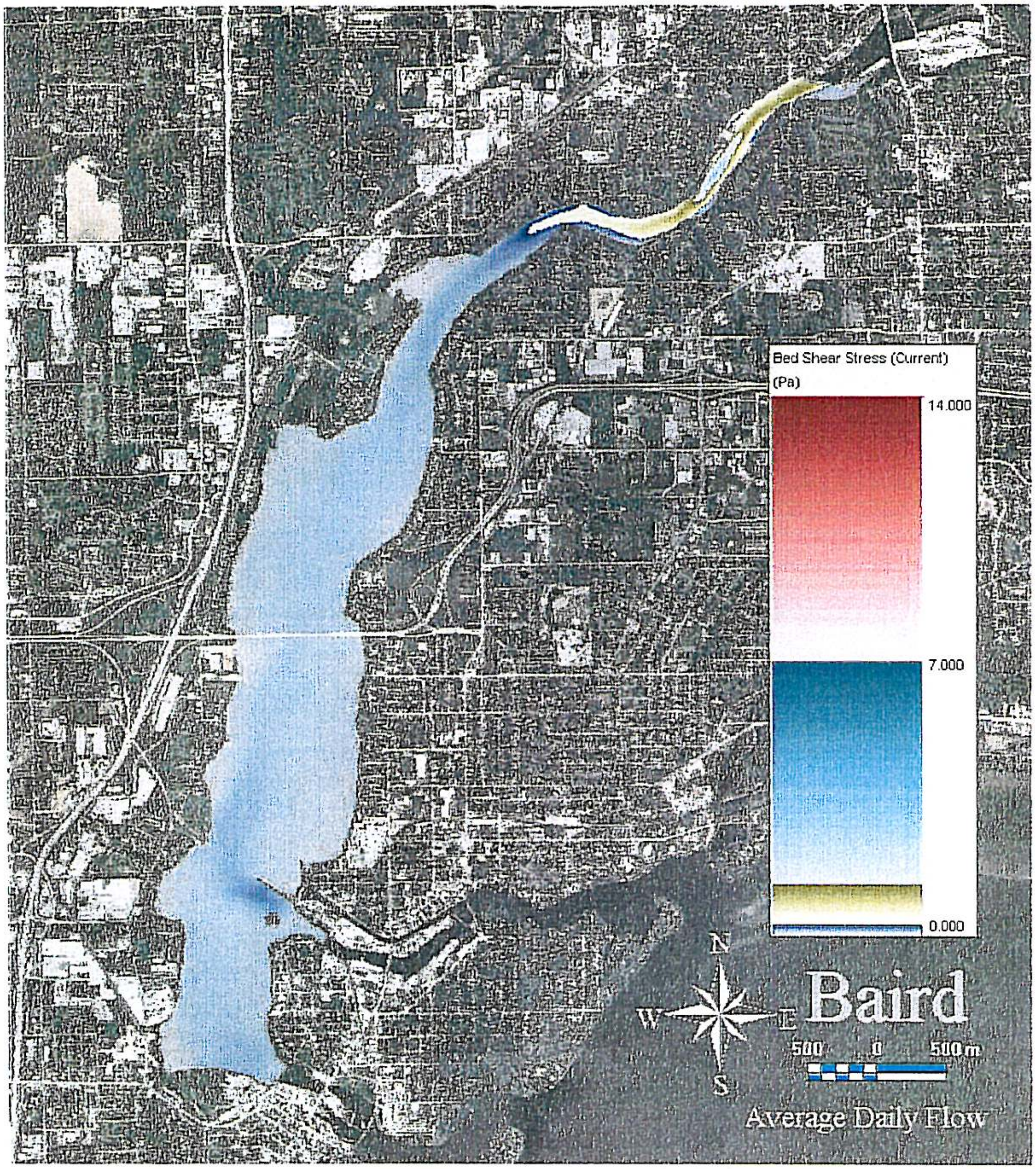

Figure 4b. Runs 4 and 5 – River Currents Only Shear Stress Map

The low shear stresses through the main part of Little Lake Butte des Morts (OU1) are a result of the fact this is a relatively wide lake where the flow is diffused.

Specifically through the proposed cap areas the shear stresses are in the range of 0.13 and 3.46 dynes/cm^2 for the 2-year return period flow, and between 0.04 and 0.65 dynes/cm^2 for the daily average flows. A summary table is presented in Table 5, and a statistical plot is shown in Figure 5.

Table 5
Bed Shear Stresses in cap areas for River Flow only model results

Flow Selection	Bed Shear Stress (Dynes/cm^2)							
	Cap 13				Cap 16			
	Min	Max	Mean	Std Dev	Min	Max	Mean	Std Dev
2-Year Return (Runs 1-3)	0.13	3.46	0.77	0.473	0.35	1.36	0.84	0.196
Daily Avg. (Runs 4-5)	0.04	0.65	0.18	0.179	0.16	0.32	0.22	0.040

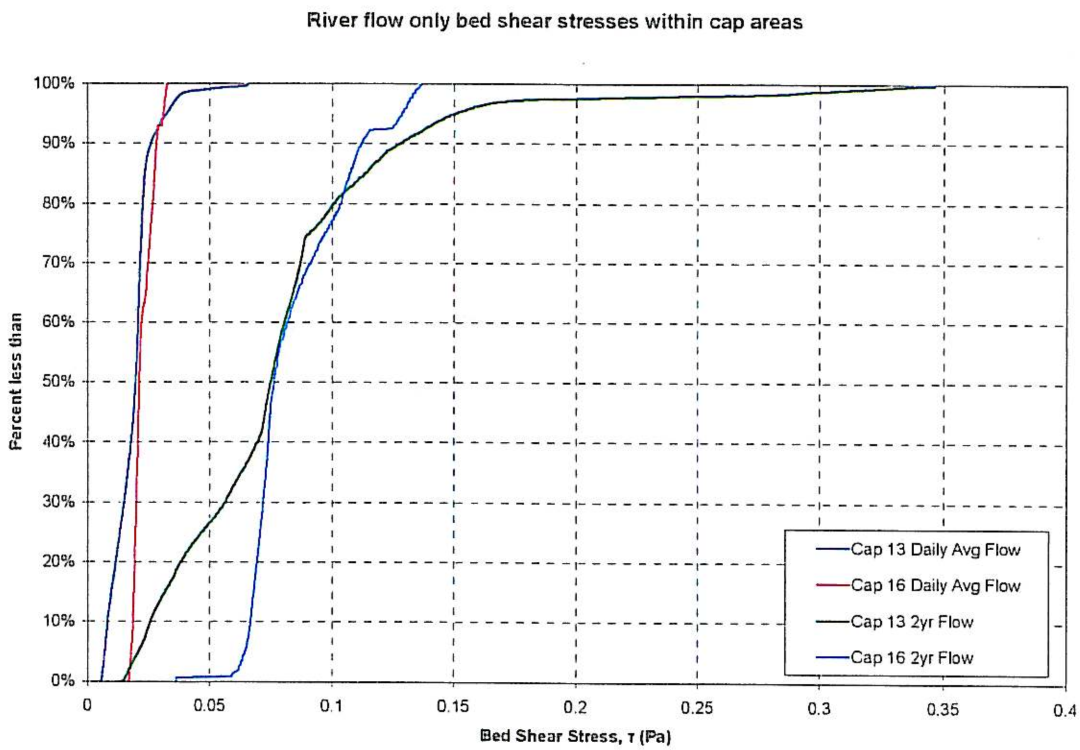

Figure 5. Bed shear stresses within cap areas for River flow only model results

3.2 Shear Stresses Generated by Wind-Waves and Wind-Driven Currents

The shear stress maps for the wave alone condition for the five model runs are shown in Figures 6a to 6e. As expected the most widespread high shear stresses are encountered in Runs 1 and 4, and these two results are quite similar (the wind speeds are not that different, Run 1 was 52.8 mph and Run 4 was 44.1 mph, both from the SSW). For these two runs the wave-only shear stresses within the proposed cap areas were always less than approximately 3.5 Pa or 35 dynes/cm^2. The NNE wind event of Run 5 produced wave-only shear stresses in similar range as Runs 1 and 4, but as expected, more towards the south end of Little Lake Butte des Morts. The WSW and NNW wind directions of Runs 2 and 3 respectively produced the highest wave-only shear stresses on the east side of the lake and generally less than 1.3 Pa.

3.2.1 Wind-Driven Circulation

Wind-driven circulation was predicted for the Run 1 condition only using ECOMSED. Figure 7a shows the predicted flow vectors for Run 1 with the 50-year SSW wind-generated currents combined with the 2-year river flow. The flow velocities of Figure 7a can be compared to those presented in Figure 3b for the 2-year river flow alone. It is evident that flow speeds along the shallower edges of the river have been significantly increased by the addition of the wind-driven circulation. Figure 7b shows the combined river and wind-generated flow shear stress map, and again, shear stresses are increased along the edges of the river, although remain less than about 1 Pa (10 dynes/ cm^2).

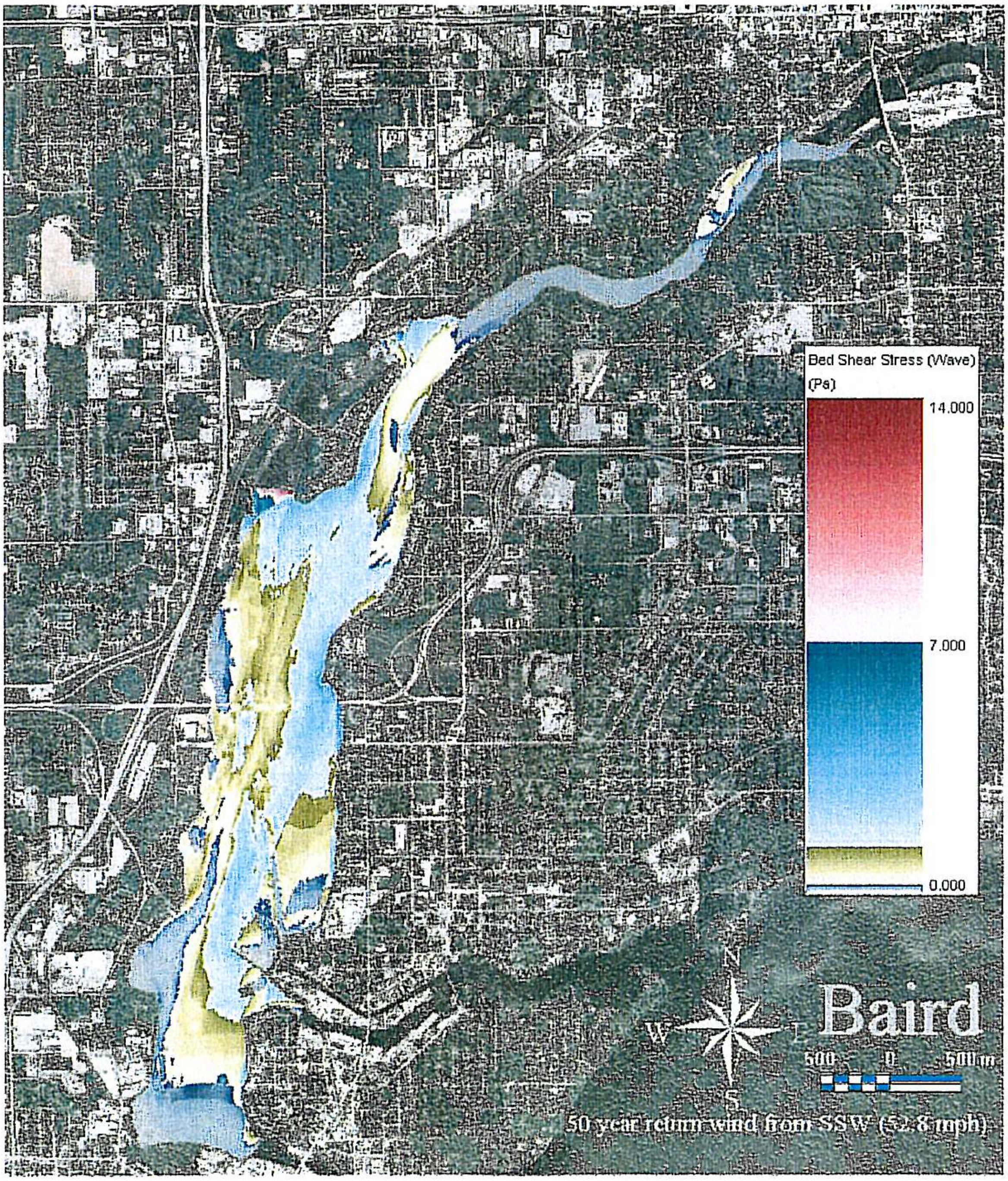

Figure 6a. Run 1 - Waves Only Shear Stress Map

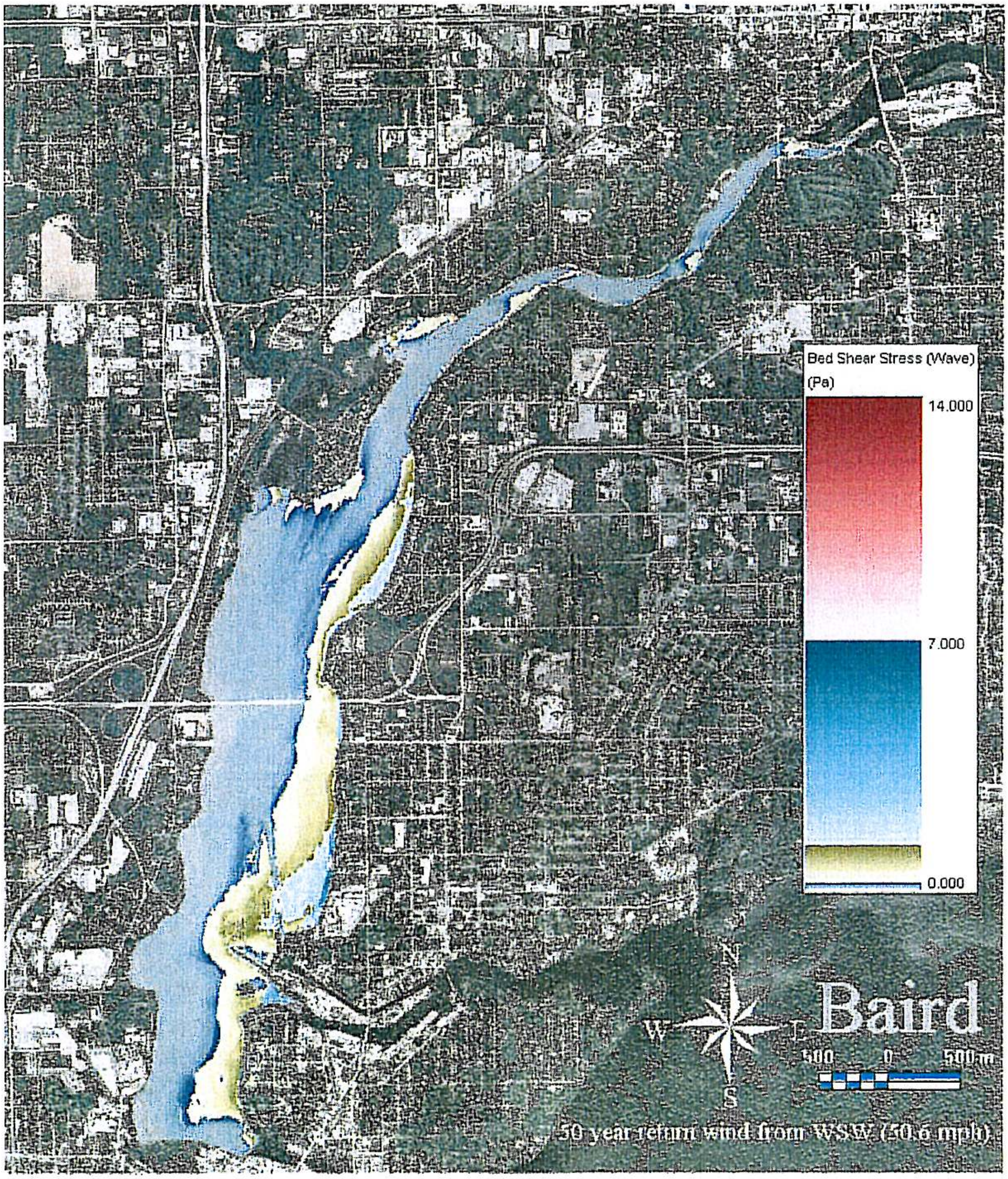

Figure 6b. Run 2 - Waves Only Shear Stress Map

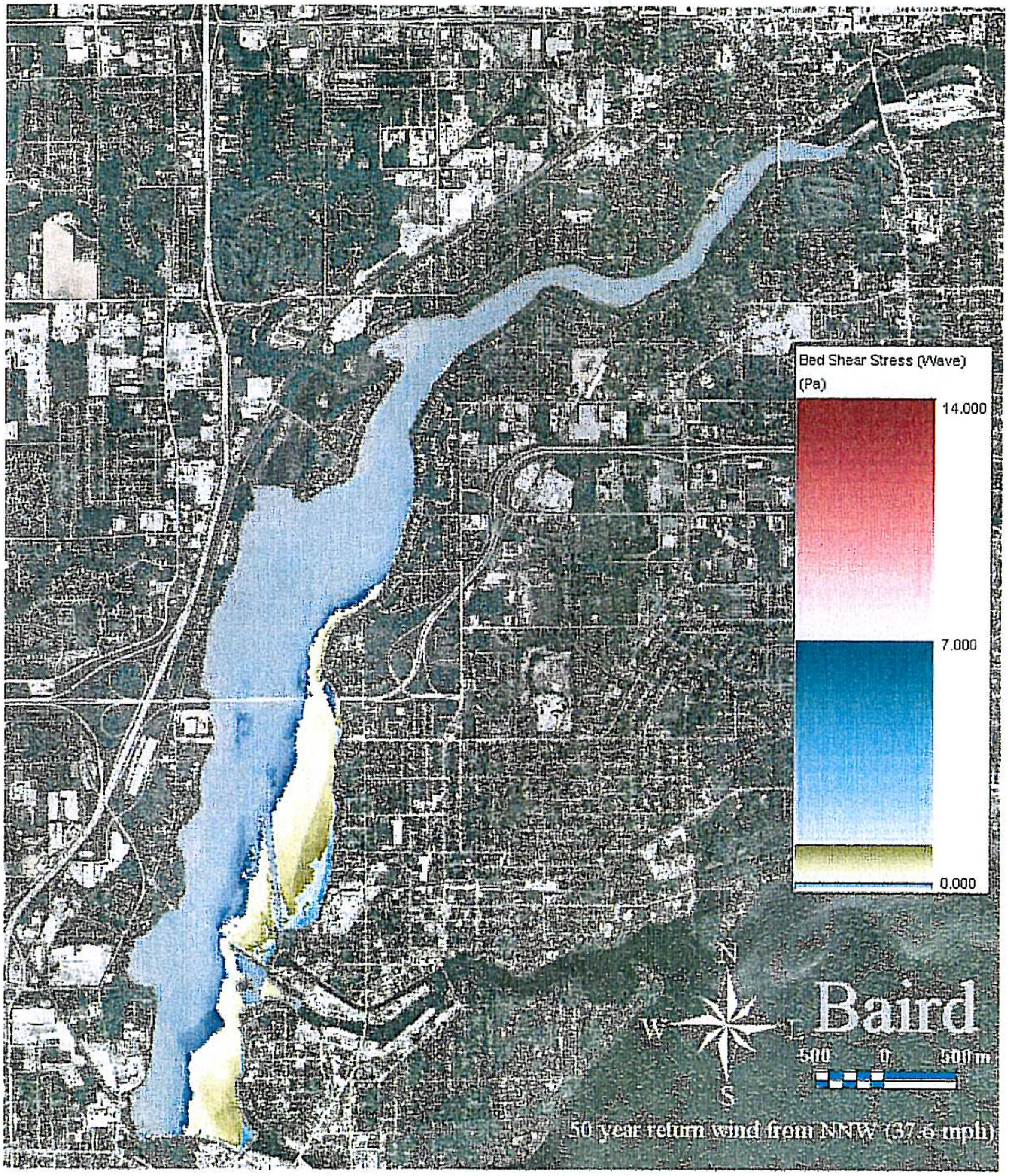

Figure 6c. Run 3 - Waves Only Shear Stress Map

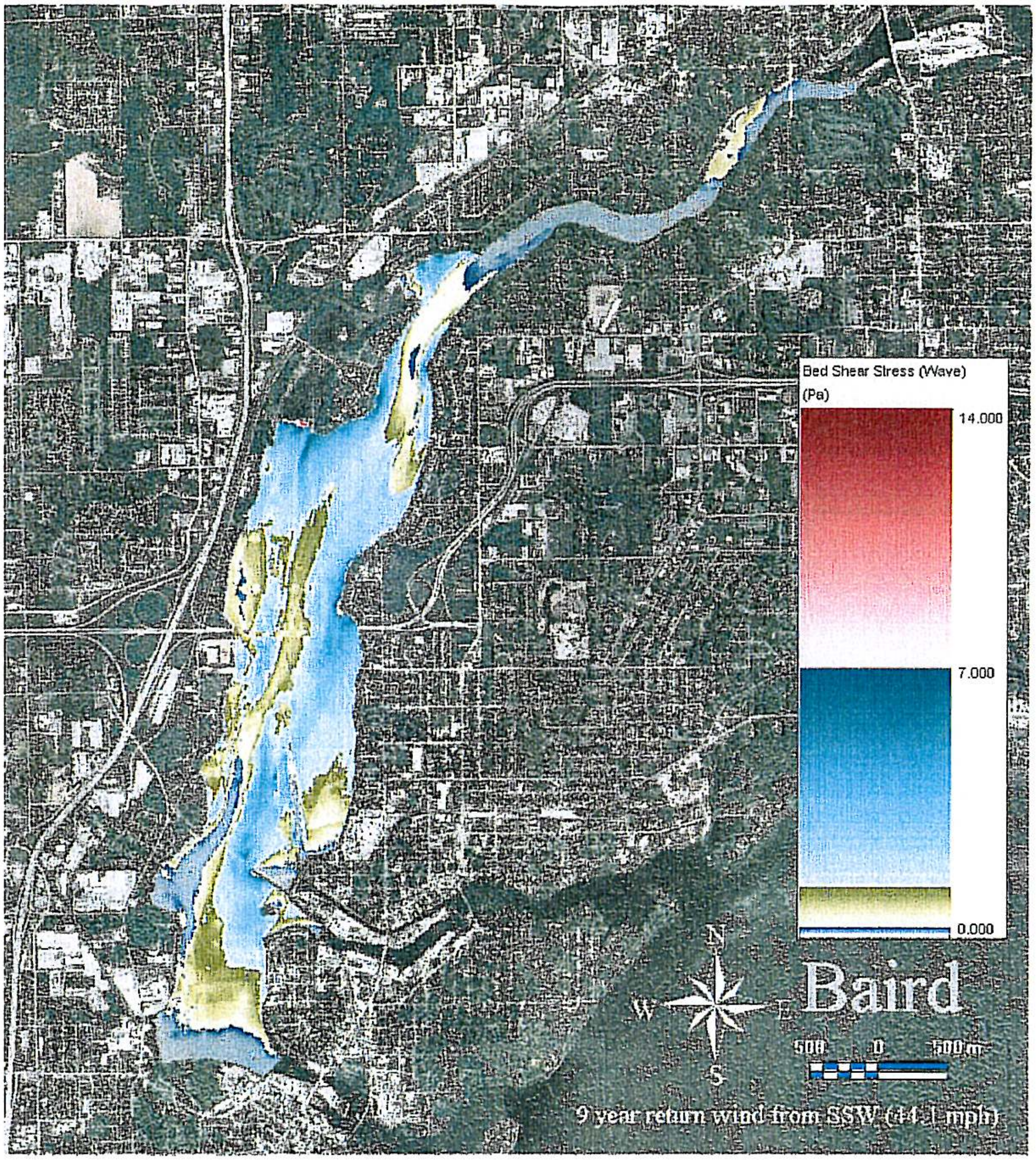

Figure 6d. Run 4 - Waves Only Shear Stress Map

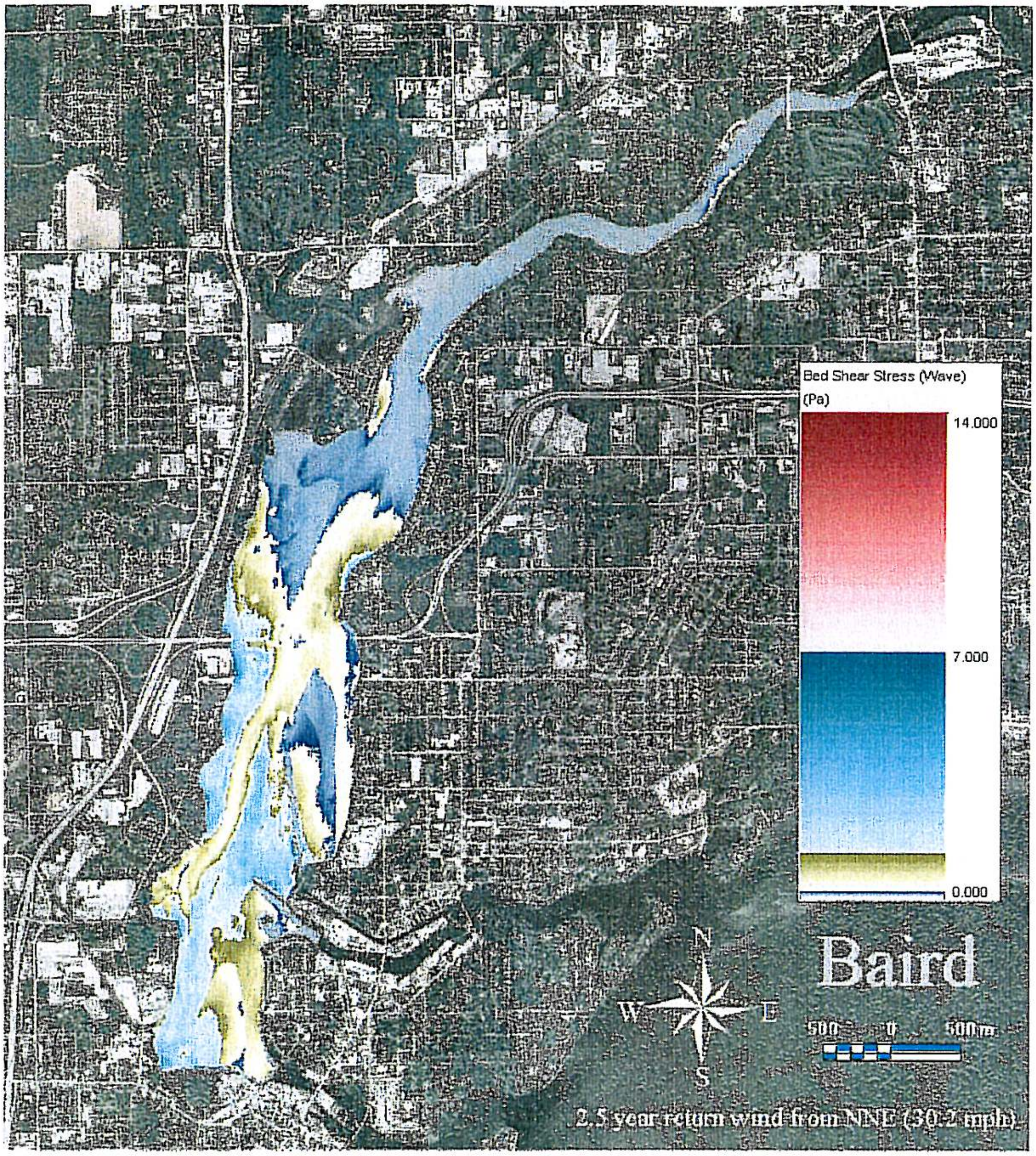

Figure 6e. Run 5 - Waves Only Shear Stress Map

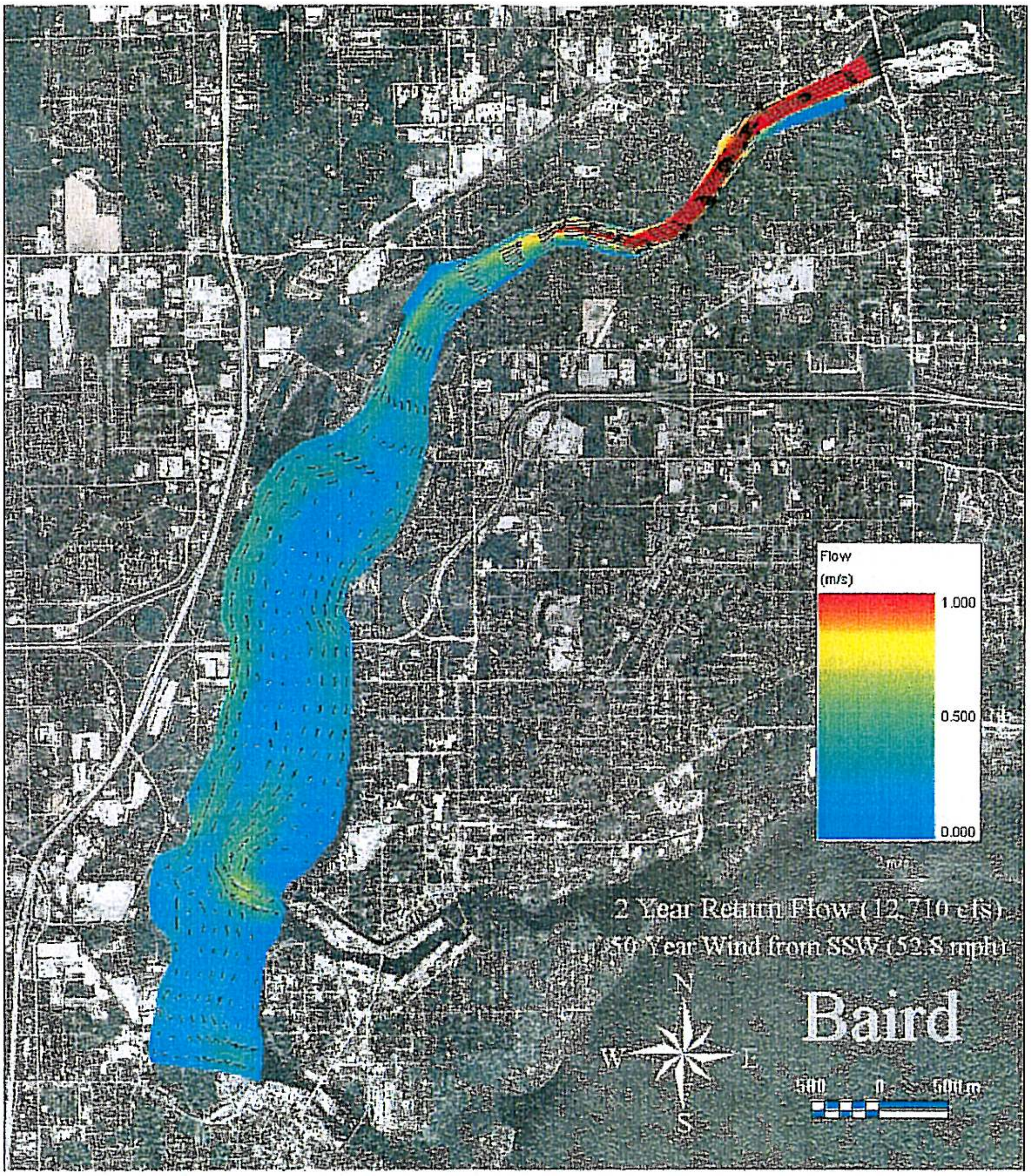

Figure 7a. Run 4 - Flow Speeds from ECOMSED for Wind and River Flow Generated Currents

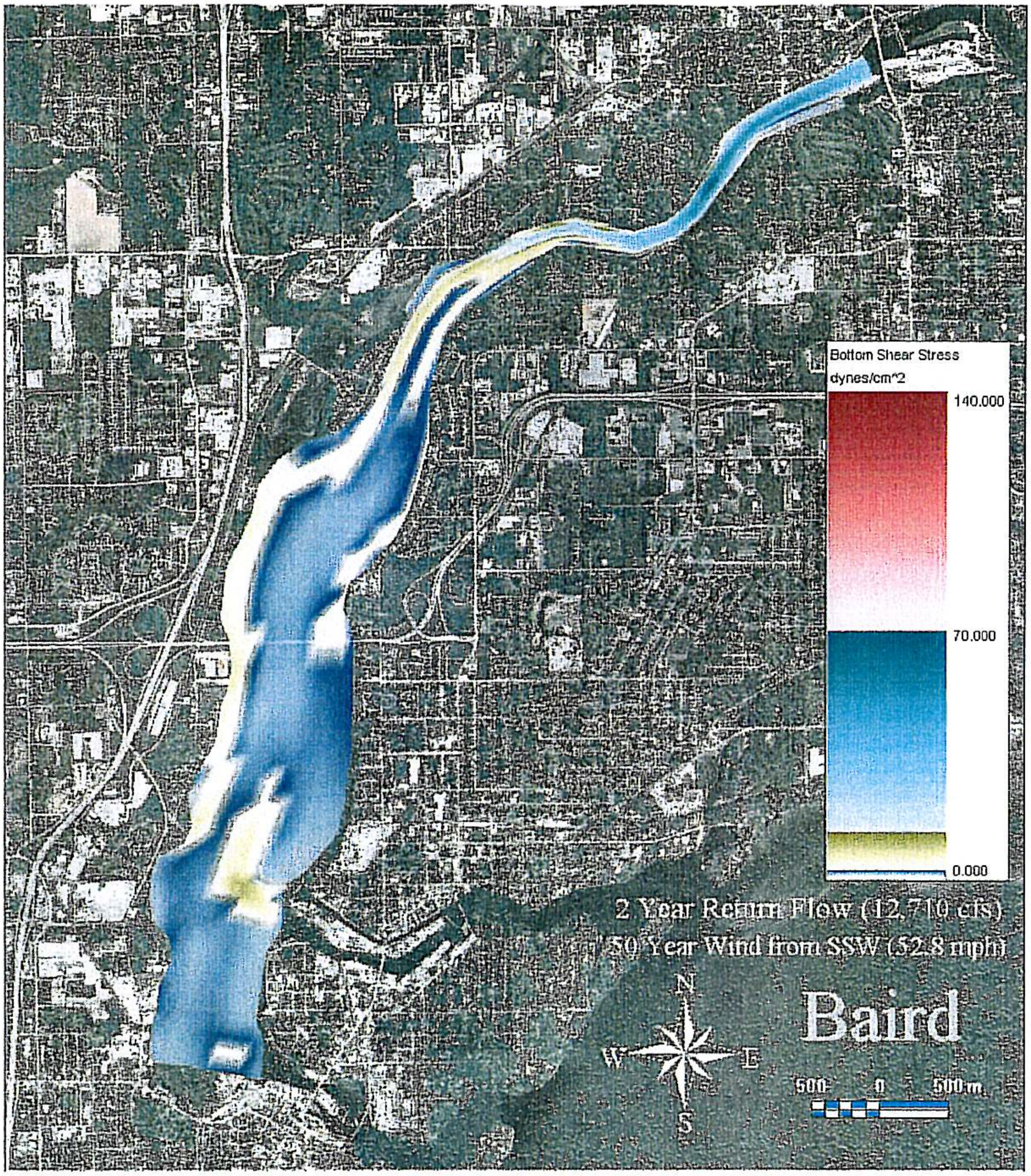

Figure 7b. Run 4 - River Flow Plus Wind-Driven Flow Shear Stress Map (i.e. no wave influence)

Numerically, the results indicate that the wind waves are more dominant than the river flow. For the runs with the wind waves only, the bed shear stresses were generally less than 3.0 Pa (30 dynes/cm^2) through the cap areas, however in an isolated area of Run 4, the maximum was up to 4.8 Pa (48 dynes/cm^2). A summary table of the results through the Cap 13 and 16 areas is provided in Table 6, and statistically presented in Figure 8.

Table 6
Bed shear stresses in cap areas for wind wave only events

Dataset	Bed Shear Stress in Cap 13 (Pa)				Bed Shear Stress in Cap 16 (Pa)			
Wind Wave Cond.	Min	Max	Mean	Std Dev	Min	Max	Mean	Std Dev
50 yr SSW (Run 1)	0.04	3.64	1.10	0.520	0.74	1.99	1.50	0.215
50 yr WSW (Run 2)	0.00	0.55	0.04	0.062	0.01	0.13	0.06	0.019
50 yr NNW (Run 3)	0.00	0.49	0.02	0.040	0.00	0.01	0.00	0.001
9 yr SSW (Run 4)	0.12	4.82	1.53	0.653	0.87	2.58	1.73	0.250
2.5 yr NNE (Run 5)	0.00	3.41	0.58	0.608	0.00	0.27	0.06	0.056

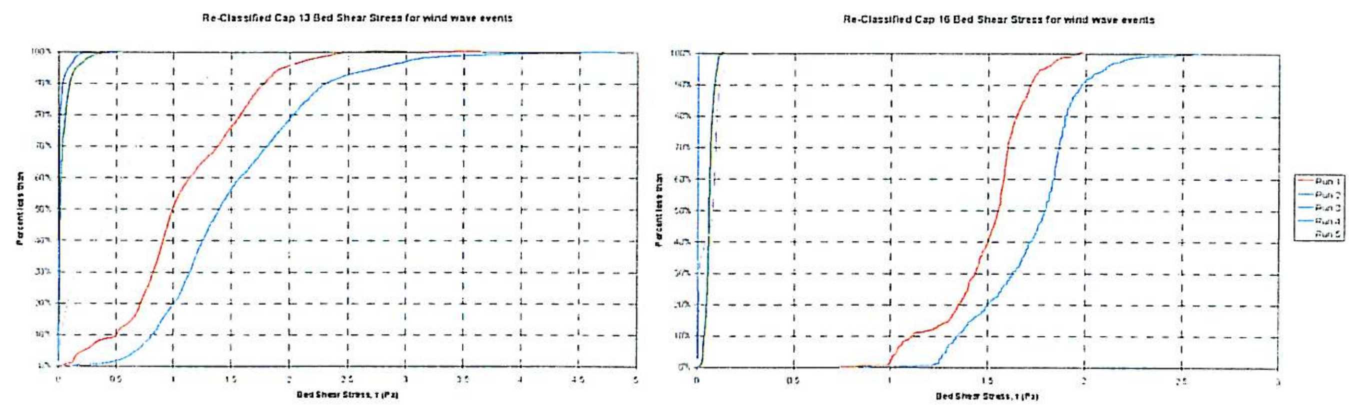

Figure 8. Bed shear stresses within cap areas for wind-wave event results only

It is worth noting that the bed shear stresses for Run 4 are greater than those in Run 1 despite the fact that the Run 4 condition included a lower wind speed event. On the basis of the equations described in Section 2.7, the bed shear stress generated by a combined wave and current climate is a function of wave related variables (height, period, and direction), flow velocity (speed and direction), and water depth. The wave information was calculated using STWAVE while the flow velocity and water depth was extracted from the ECOMSED model. Looking at the wave-alone case, the wave-induced bed shear not only depends on the wave height and period, but also on the water depth. The bed shear stress induced by waves decreases greatly as water depth increases (this is because the bottom orbital velocities decrease with increasing depth). The ECOM model result shows that the water level in Little Lake Butte des Morts depends upon the flow conditions from Lake Winnebago. The water levels in Little Lake Buttes des Morts increase to about 1.0 m above LWD during the 2-year return flow event, and for a 0.2 m water level at the Appleton Dam. The water levels in the lake increase to only 0.11m above LWD during the mean-flow conditions, with a -0.1 m water level at the Appleton Dam (the latter being the assumed downstream boundary condition for Runs 4 and 5). As a result, the greater water depth for Run 1 results in less wave-induced bed shear stress than was observed in Run 4. The 50-year wind event with average flow conditions would result in higher combined shear stresses than any of the Runs 1 to 5.

3.3 Combined Wave-Current Shear Stresses Generated by Wind-Waves and River Flows

The combined shear stresses generated by wave-current interaction between the orbital velocities generated by wind-waves and the river flow are presented for the five model runs in Figures 8a to 8e (note that wind-generated currents were not considered in this analysis).

A comparison of the results of Runs 1, 4 and 5 show that the highest shear stresses occur in different areas for different combinations of events. However, for the selected events, shear stresses are almost always less than 3.5 Pa or 35 dynes/cm^2 through the cap area with a very small section of Run 4 having an estimated bed shear stress of approximately 4.8 Pa or 48 dynes/cm^2. A summary table of the results through the Cap 13 and 16 areas is provided in Table 7, and statistically presented in Figure 9.

Table 7
Bed shear stresses in cap areas for combined flow events

Dataset			Bed Shear Stress in Cap 13 (Dynes/cm^2)				Bed Shear Stress in Cap 16 (Dynes/cm^2)			
Flow Cond.	Wind Cond.	Run #	Min	Max	Mean	Std. Dev.	Min	Max	Mean	Std. Dev.
2 Year	50 yr SSW	1	0.55	36.62	11.78	5.23	7.99	20.59	15.71	2.18
2 Year	50 yr WSW	2	0.13	5.84	1.15	0.77	0.42	2.13	1.45	0.28
2 Year	50 yr NNW	3	0.13	5.25	0.92	0.59	0.36	1.36	0.84	0.19
Daily Avg.	9 yr SSW	4	1.77	48.24	15.44	6.52	8.82	25.97	17.47	2.49
Daily Avg.	2.5 yr NNE	5	0.18	34.12	5.96	6.04	0.17	2.88	0.86	0.56

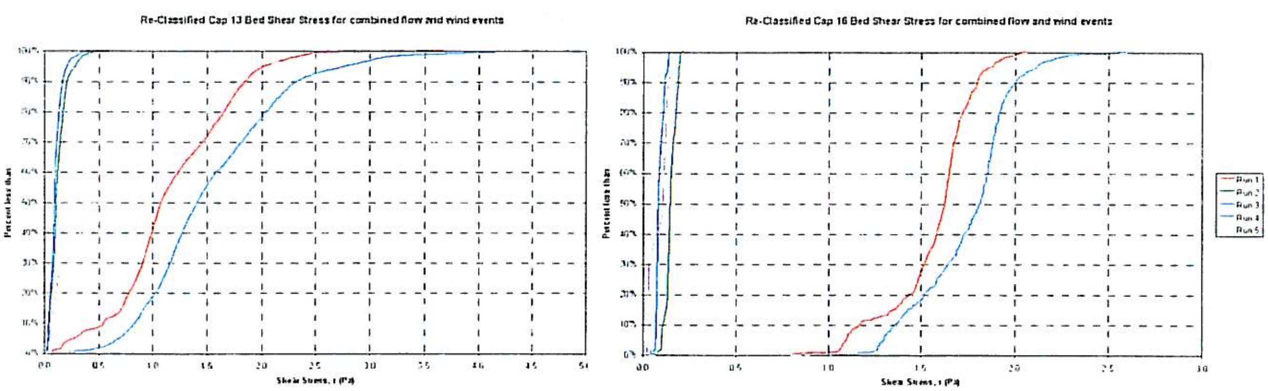

Figure 9. Bed shear stresses within cap areas for combined river flow and wind results

It is noticeable that the trend of Run 4 having higher bed shear stresses than Run 1 through the cap area continues. This is once again caused by higher water levels in Run 1. Furthermore, it demonstrates that the wave component is more dominant than the flow component, particularly with lower water levels. A run with a 50-year return period wind from the SSW combined with an average flow condition would result in higher shear stresses than any of the combinations considered in Runs 1 to 5.

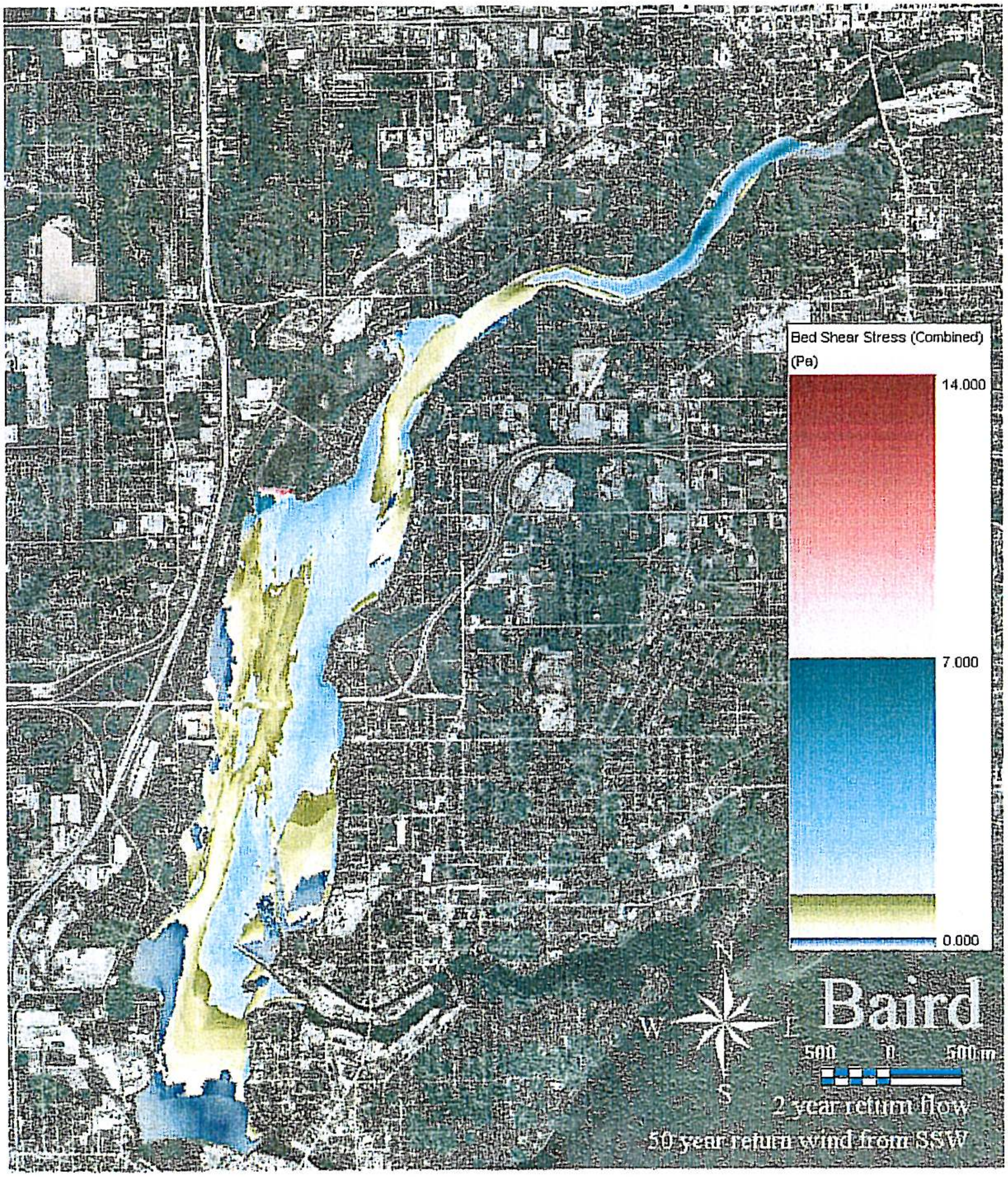

Figure 8a. Run 1 - Combined Wave-Current Shear Stress Map

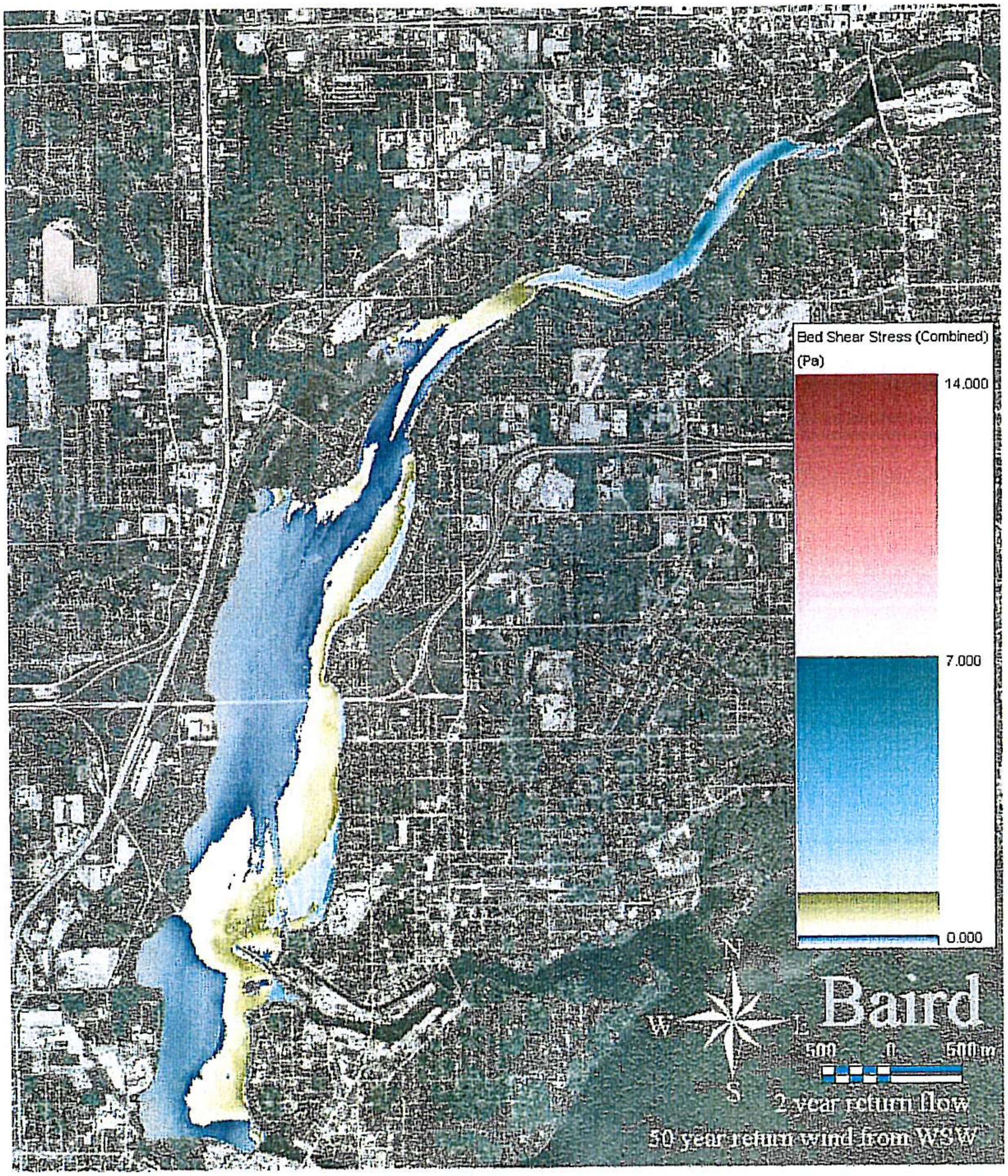

Figure 8b. Run 2 - Combined Wave-Current Shear Stress Map

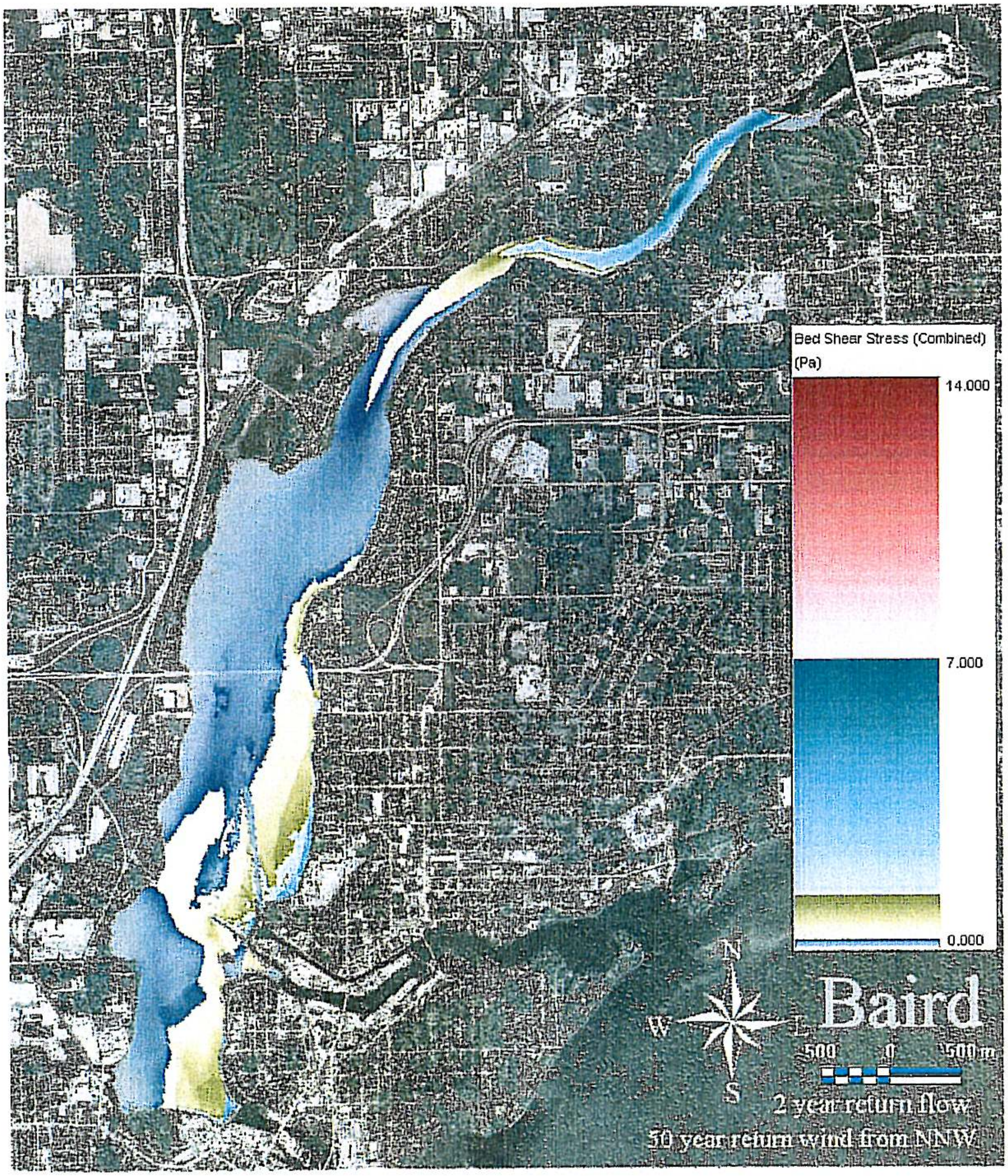

Figure 8c. Run 3 - Combined Wave-Current Shear Stress Map

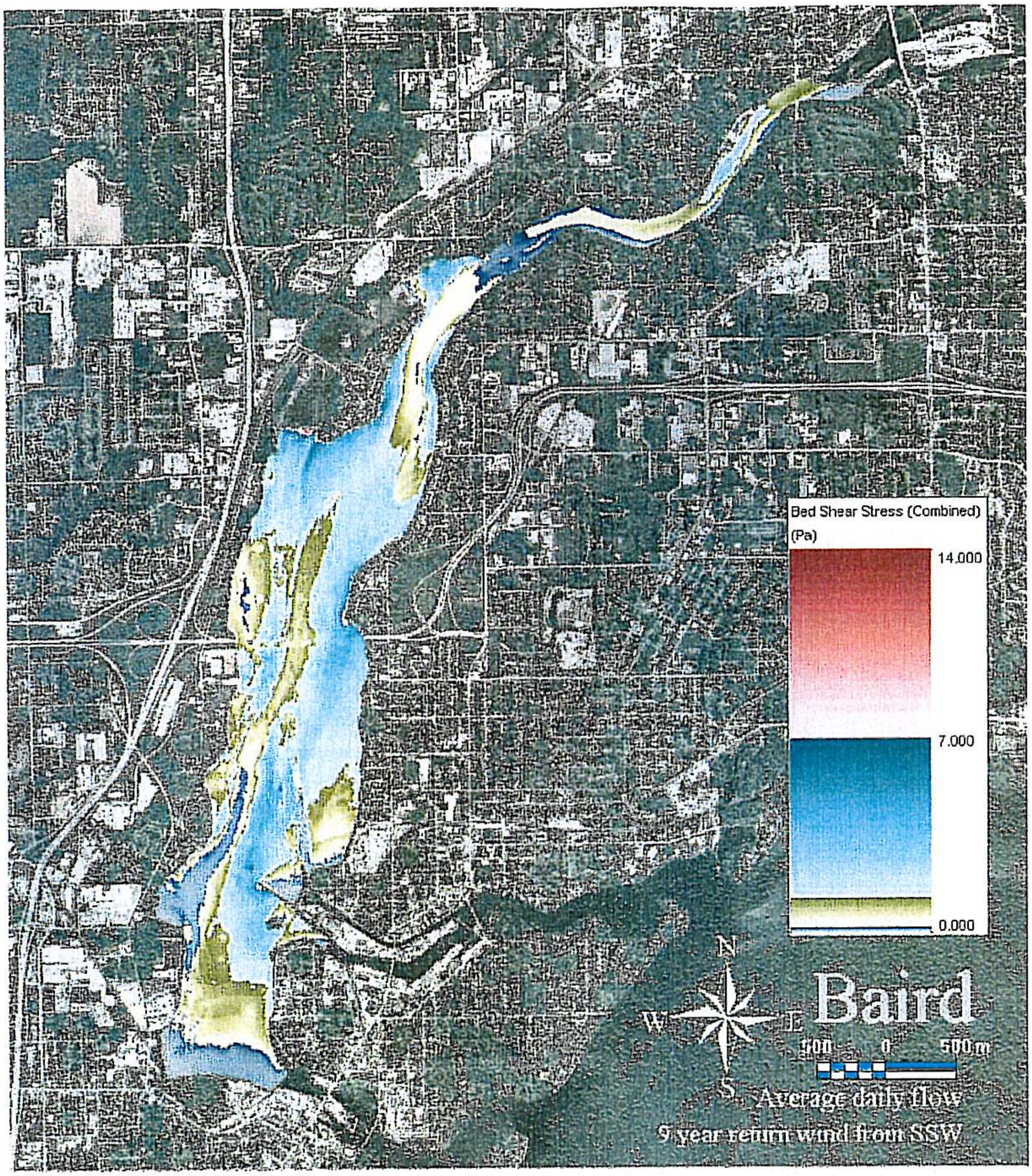

Figure 8d. Run 4 - Combined Wave-Current Shear Stress Map

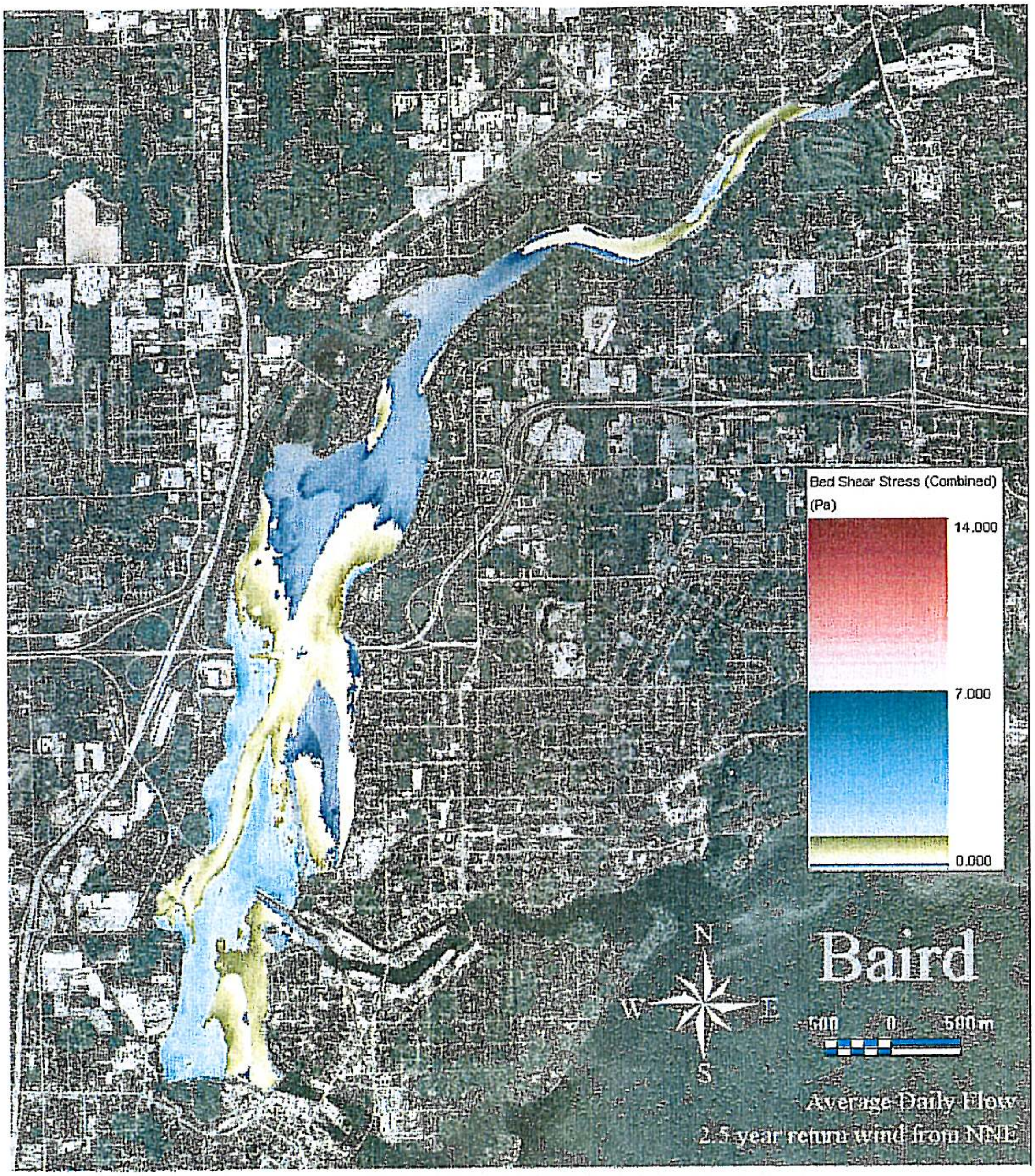

Figure 8e. Run 5 - Combined Wave-Current Shear Stress Map

4.0 REVIEW OF PAPERS ON PUMPING AND LIQUEFACTION

In order to consider other implications of wind-waves on the proposed OU1 cap, Baird reviewed relevant literature identified by Foth on the processes of wave pumping and liquefaction. These processes may affect the design of the cap for OU1. However, specific design recommendations regarding these processes were not intended to be within the scope of this work. Instead, the objective was to provide opinion on the relevance of these processes to the cap design process.

The processes of wave pumping and liquefaction are related to the pore-pressures in submarine soil. As a wave passes, the increase in pressure above the crest of the wave pumps additional pressure into the soil matrix below the crest. Sometimes, as the trough of a wave passes, there is insufficient time for the pressure to dissipate, resulting in a buildup of pressure until eventually the pressures are greater than the overburden pressure; at this point the soil acts as a fluid and may flow downhill or allow supported structural elements to sink into it. A secondary way for soils to liquefy under waves is for momentary liquefaction to occur, this is as a result of a wave trough passing: the sudden reduction in water pressure at the bottom reduces the overburden pressure sufficiently to allow a temporary liquefaction of a relatively small area of soil. Liquefaction not resulting from waves (such as earthquakes) is not considered here.

4.1 Wave Pumping Review

The two papers that were reviewed on wave pumping were:

- Habel and Bagtzoglou (2005)
- Precht and Heuttel (2003)

Habel and Bagtzoglou (2005) indicate that the flux through a porous bed is a function of the ratio of wave length to water depth and soil permeability. For the 50-year wind conditions the predicted peak wave period was approximately 2.5 s for the longer fetch events (SSW), and approximately 2.2 s for the shorter fetch events (WSW). The average water depth in the areas of interest is in the range of 2 m. These conditions give a wave length of 8.7 m for the SSW events and 7.2 m for the WSW events. These correspond to a wave length to water depth ratio of 4.35 and 3.55, respectively. If one assumes a permeability for the proposed cap corresponding to sandy gravel ($K = 1 \times 10^{-11}\ m^2$), the potential flux at the bed interface may be estimated from Figure 5 of Habel and Bagtzoglou (2005). This approach yields fluxes in the range of approximately 4 to 8 x 10^{-6} m/s. It is likely there will be flux between the river bed and the underside of a 13-inch fine to medium gravel cap as currently envisaged, however filter layers could be used to minimize the risk of it occurring. More detailed consideration of potential flow between the native sediment and the overlying water would require information on the anticipated filtering design and more detailed analysis is beyond the scope of this review. The Precht and Heuttel (2003) paper primarily focuses on conditions where ripples are created on the surface of a sandy deposit. This will likely not occur for the proposed cap of OU1 as the material will be selected to be stable under all conditions.

4.2 Liquefaction Review

The documents reviewed on liquefaction were:

- Nataraja and Gill (1983)
- Madsen (1978)
- De Wit (1995)
- De Wit and Kranenburg (1997)

Wave-induced liquefaction is caused by either temporal or spatial gradients in pore-water pressures within the matrix of a soil generated by wave-induced pressures. Liquefaction can result in the loss of soil strength, resulting in a flow of the soil mass down a slope, or supported structures sinking into the weakened sediment. Although the waves on Little Lake Butte des Morts are not large, the oscillations in wave pressures could result in liquefaction that would allow the cap armor to sink into the supporting soft sediments. Madsen (1978) presents an equation to evaluate the potential for failure conditions in cohesionless sediment. For a water depth of approximately 2 m, the 50-year wind event resulted in a wave period of 3 s and wave height of 60 cm in the cap zone; for this wave condition it is clear that liquefaction failure of coarse sandy sediment is not possible, and certainly not possible for an anticipated cap sediment size of greater than 25 mm. Furthermore, in a case where there are no foundations of concern, liquefaction only becomes an issue where there is sufficient slope of the material enabling it to be transported away from its initial position. This requires relatively steep slopes such that auto-suspension and/or underwater avalanching conditions can be generated (Nairn, 1990). For sandy beds (i.e. with a median grain size in the range of 0.2 mm) this would require slopes significantly steeper than approximately 1 (vertical): 2.5 (horizontal) together with a flow speed of approximately 1 m/s. This range of bed slopes and flow speeds do not exist in OU1, and the anticipated cap sediment size is much coarser. Possible interaction between wave-induced pore-water flows and the native material under the cap may be another issue to consider, however, this is more a question of appropriate filtering design between the layers.

De Wit and Kranenburg (1997) studied liquefaction in cohesive sediments caused by waves. They undertook a series of physical modeling studies, with the primary emphasis on China, Caland, and Westwald clays. They found that liquefaction occurs in these clayey materials when the pore pressure reaches approximately the soils yield stress value. Following from this, they observed that the required wave height to induce liquefaction increased with consolidation time. This means that after placing the cap, the probability of liquefaction occurring decreases with time, down to some minima. The level of consolidation that occurs as a result of placing the cap is dependant upon the soil type and properties beneath the cap and the final cap design. Therefore, consideration of the potential for liquefaction of the cohesive sediment underlying the cap is dependent on the change of the cohesive soil characteristics associated with the construction of the cap, and specifically, the degree of consolidation. This type of geotechnical assessment is beyond the scope of this preliminary review.

5.0 CONCLUSIONS AND RECOMMENDATIONS

The conclusions of this report include and build on the conclusions and recommendations of the Baird (2006) report that evaluated river-current only shear stresses for the 100-year return period flow event.

- Baird (2006) found that the bed shear stresses in Little Lake Butte des Morts (Fox River OU1) under the condition of the 100-year return flow are generally less than 1.3 Pa (13 dynes/cm^2) and this is associated with the critical shear stress for erosion for very coarse sand (< 2 mm). This indicates that very coarse sand is likely stable under the 100-year return river flow alone (i.e. no waves or wind) for areas with shear stress equal to or less than 13 dynes/cm^2.
- Baird (2006) also found that there is no significant impact of water level change, at the Appleton Dam, on the bed shear stress generated by river flows in Little Lake Butte des Morts.
- The current study evaluated selected combinations of possible combined 100-year (events) return period conditions associated with river currents, orbital velocities generated by wind-waves, and to a limited extent, steady flows generated by wind stresses.
- A wind record and extreme value analysis was completed with winds from the Green Bay and Appleton wind stations. The analysis determined that there was limited or no dependency between extreme flows and extreme winds. Therefore, the 100-year combined event can be approximated as a direct multiple of wind speed return period and river flow return period (e.g. 50-year wind and 2-year flow or 33-year flow and 3-year wind).
- Three possible 100-year conditions were selected consisting of 50-year return period winds from the SSW, WSW and NNW combined with the 2-year return period river flow condition. The US Army Corps of Engineers STWAVE model was applied to predict the wave conditions for the selected wind events on OU1.
- Two additional runs were completed to evaluate two conditions similar to the approach used to derive the wind-wave and flow design shear stresses for OU2 to OU5 (Shaw and Anchor, 2006). Through the cap areas, the event from the SSW resulted in comparable bed shear stresses as the SSW 100-year return period events.
- Combined shear stresses were calculated using the approach of Van Rijn (1993) and it is noted that, in conditions where waves and currents are occurring simultaneously, it is necessary to compute the combined shear stresses considering wave-current interaction in the boundary layer.
- For the selected conditions that were evaluated, the combined wave-current shear stresses were found to be almost everywhere less than 3.5 Pa (35 dynes/cm^2) through both cap areas 13 and 16, with the maximum predicted shear stress of 4.8 Pa (48 dynes/cm^2). The mean values in the cap areas were at most 1.7 Pa (17 dynes/cm^2) for the SSW conditions, other directions had lower bed shear stresses. The 90th and 95th percentiles were at most 2.3 Pa (23 dynes/cm^2) and 2.8 Pa (28 dynes/cm^2), respectively. Throughout both the Cap 13 and 16 areas, the predicted maximum bed shear stresses correspond to the stability of a coarse sand or fine gravel. The GIS files for all of the shear stress maps have been provided with this report (see Section 3).
- It is noted that there may be other 100-year return period combinations of wind speed and flow conditions that generate higher shear stresses, and therefore, represent the true 100-year bed shear stress condition. On this reach of the Fox River, owing to the width of the lake and the diffused river flow, wave-generated shear stresses dominate. Also, as determined through this analysis, lower flow

conditions together with high wind speeds result in the highest bed shears stresses due to the greater influence of wave-generated shear stresses for Little Lake Buttes des Morts and due to the significant influence of shallow water (associated with low flows) on increasing wave-driven shear stresses. Therefore, it is possible the 100-year wind condition with an average flow condition may result in the true 100-year shear stress condition. It is also possible that different parts of the cap may experience a 100-year shear stress condition from different wind conditions. The two most severe wind conditions for the cap area appear to be SSW (for the central and north end) and NNE (for the south end).

- The influence of wind-generated circulation on the flows in Little Lake Butte des Morts was predicted with ECOMSED for Run 1 with currents alone. This simulation showed that although flow speeds and related shear stresses vary both spatially and temporally through the model runs, they are generally significantly increased along the 200 to 300 m wide band at the edge of the river. However, the wind-generated flow shear stresses remain less than 1 Pa (10 dynes/cm^2). This influence was ***not*** included in the combined wave-current shear stress estimate for Run 1. If additional runs are completed with higher wind speeds (such as the 100-year wind speed), it would be advisable to consider the influence of wind-generated currents, particularly for analysis in shallow areas where the impact is much greater.
- Liquefaction concerns will be mitigated to some extent through the consolidation of the underlying sediment associated with the loading that will occur with placement of the cap. The extent of consolidation associated with placement of the cap would need to be evaluated by a geotechnical engineer. Wave-pumping was also reviewed in a preliminary manner and it was determined that there could be flow extending down below a medium-sized gravel cap thickness of 13 inches as currently envisaged. Details on filtering between the cap and the underlying native sediment would need to be considered in a more detailed evaluation of both liquefaction potential and wave pumping.

6.0 REFERENCES

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USACE (2001). "STWAVE: Steady-State Spectral Wave Model", *User's Manual for STWAVE*, Version 3.0. Coastal and Hydraulic Laboratory, ERDC.

Van Rijn, L.C. (1993). Principles of Sediment Transport in Rivers, Estuaries and Coastal Seas. Aqua Publications, Amsterdam, The Netherlands.

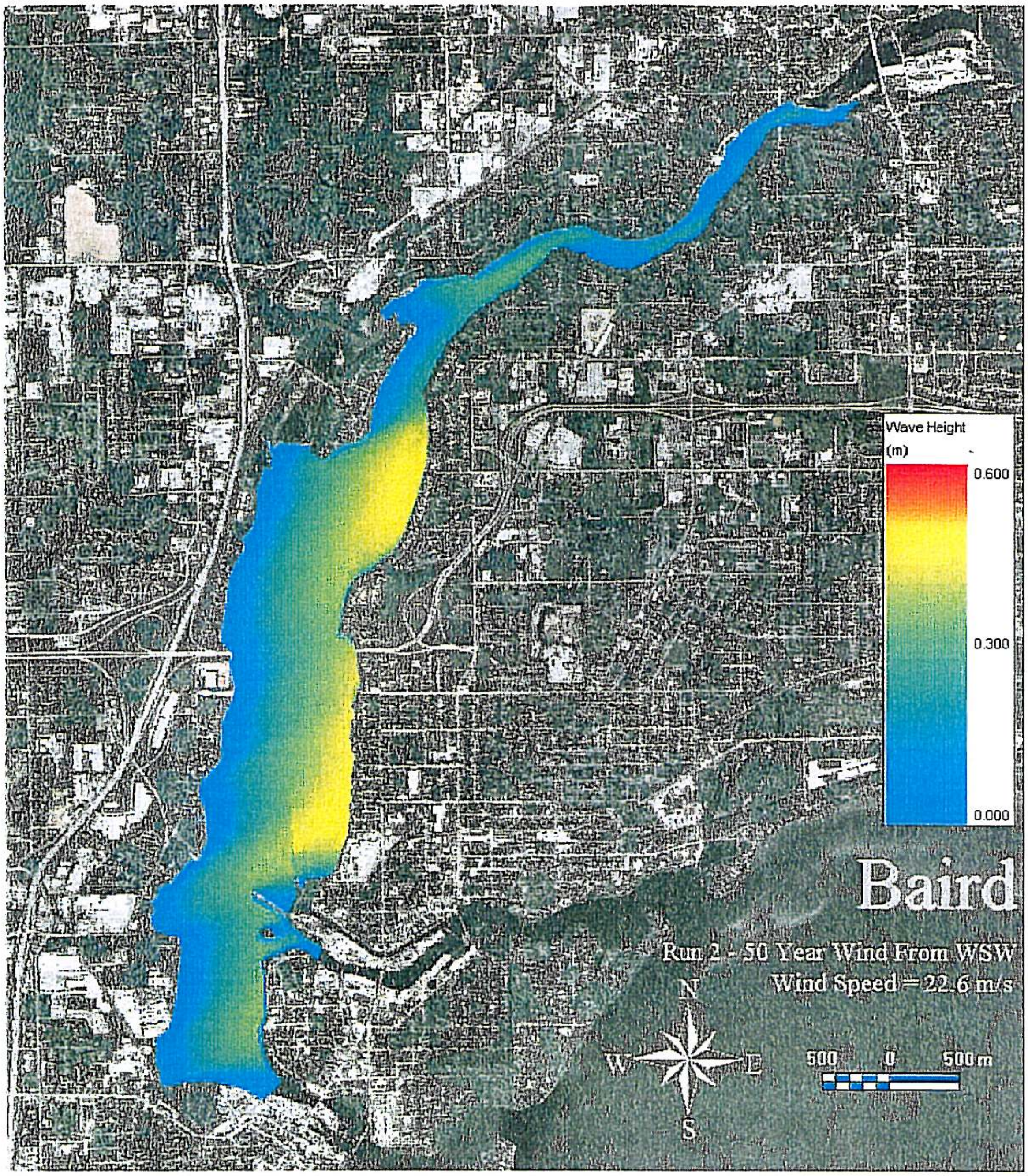
7.0 DELIVERABLES

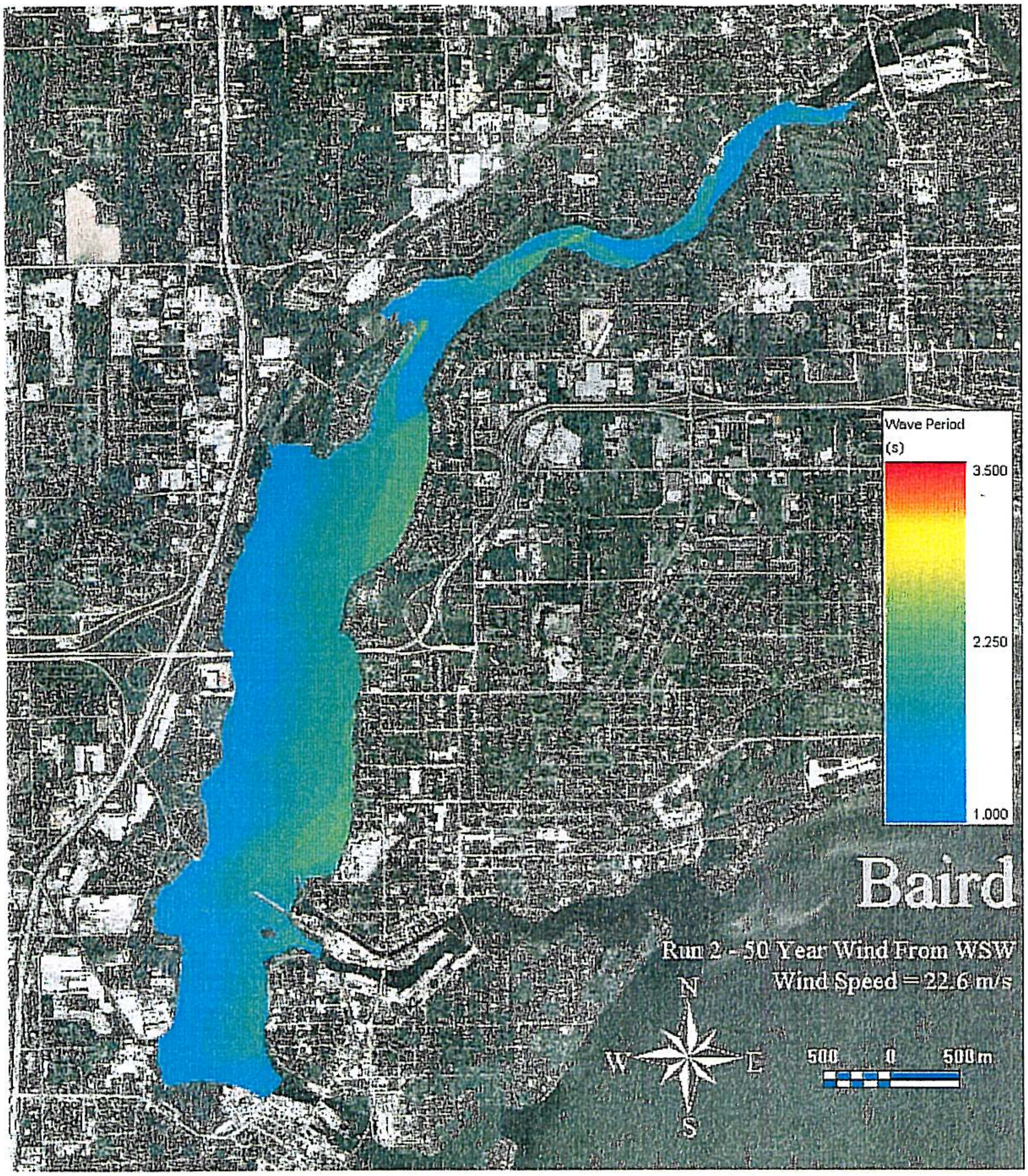
- Current Only Bed Shear Stress for a 2 year return flow (Runs 1 to 3).shp
- Current Only Bed Shear Stress for average daily flow (Runs 4 and 5).shp
- Wave Only Bed Shear for 50 year return wind from SSW (Run 1).shp
- Wave Only Bed Shear for 50 year return wind from WSW (Run 2).shp
- Wave Only Bed Shear for 50 year return wind from NNW (Run 3).shp
- Wave Only Bed Shear for 9 year return wind from SSW (Run 4).shp
- Wave Only Bed Shear for 2.5 year return wind from NNE (Run 5).shp
- Bed shear stress for combined 2 year return flow and 50 year return wind from SSW (Run 1).shp
- Bed shear stress for combined 2 year return flow and 50 year return wind from WSW (Run 2).shp
- Bed shear stress for combined 2 year return flow and 50 year return wind from NNW (Run 3).shp
- Bed shear stress for combined daily average flow and 9 year return wind from SSW (Run 4).shp
- Bed shear stress for combined daily average flow and 2.5 year return wind from NNE (Run 5).shp

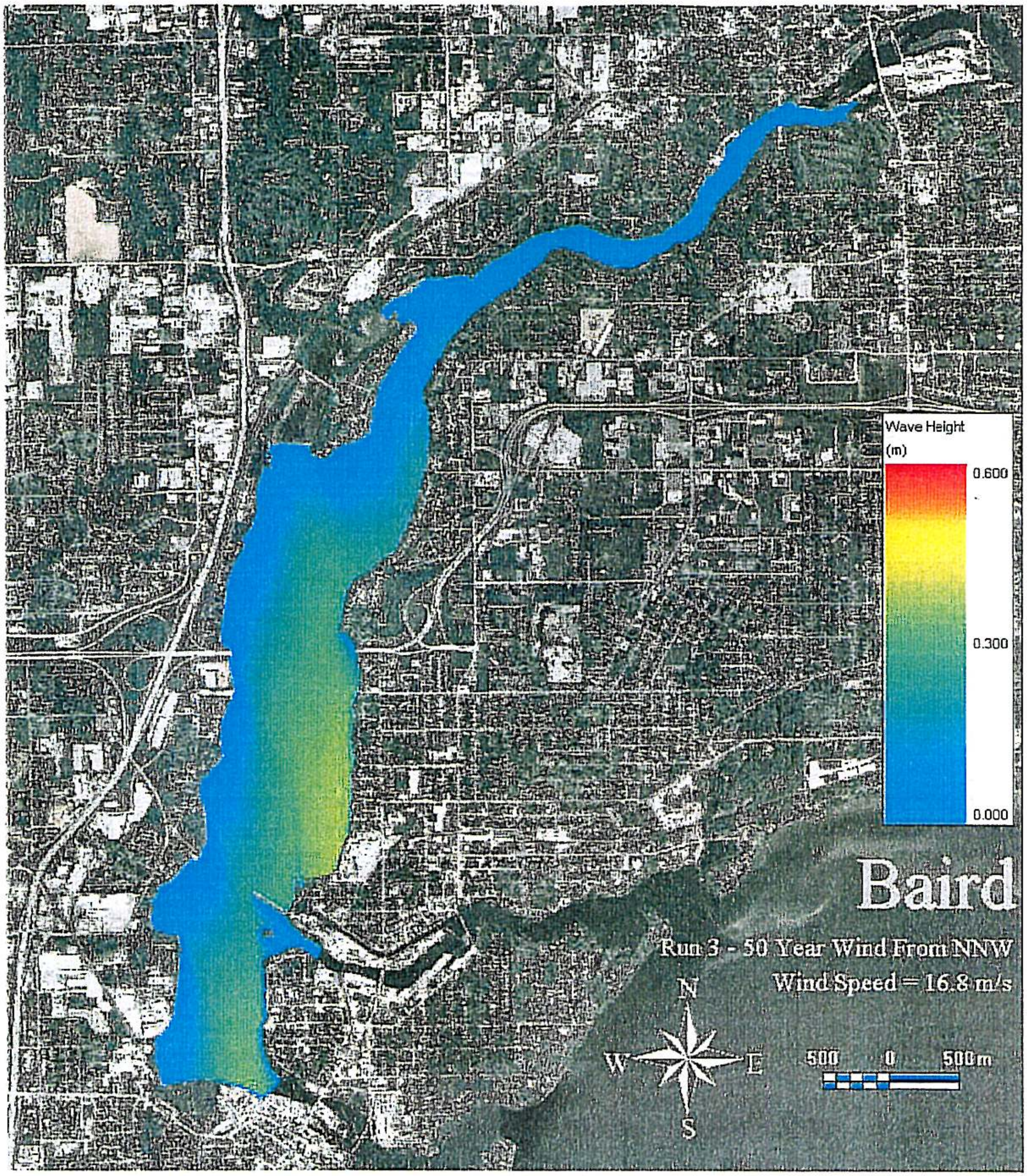
APPENDIX A

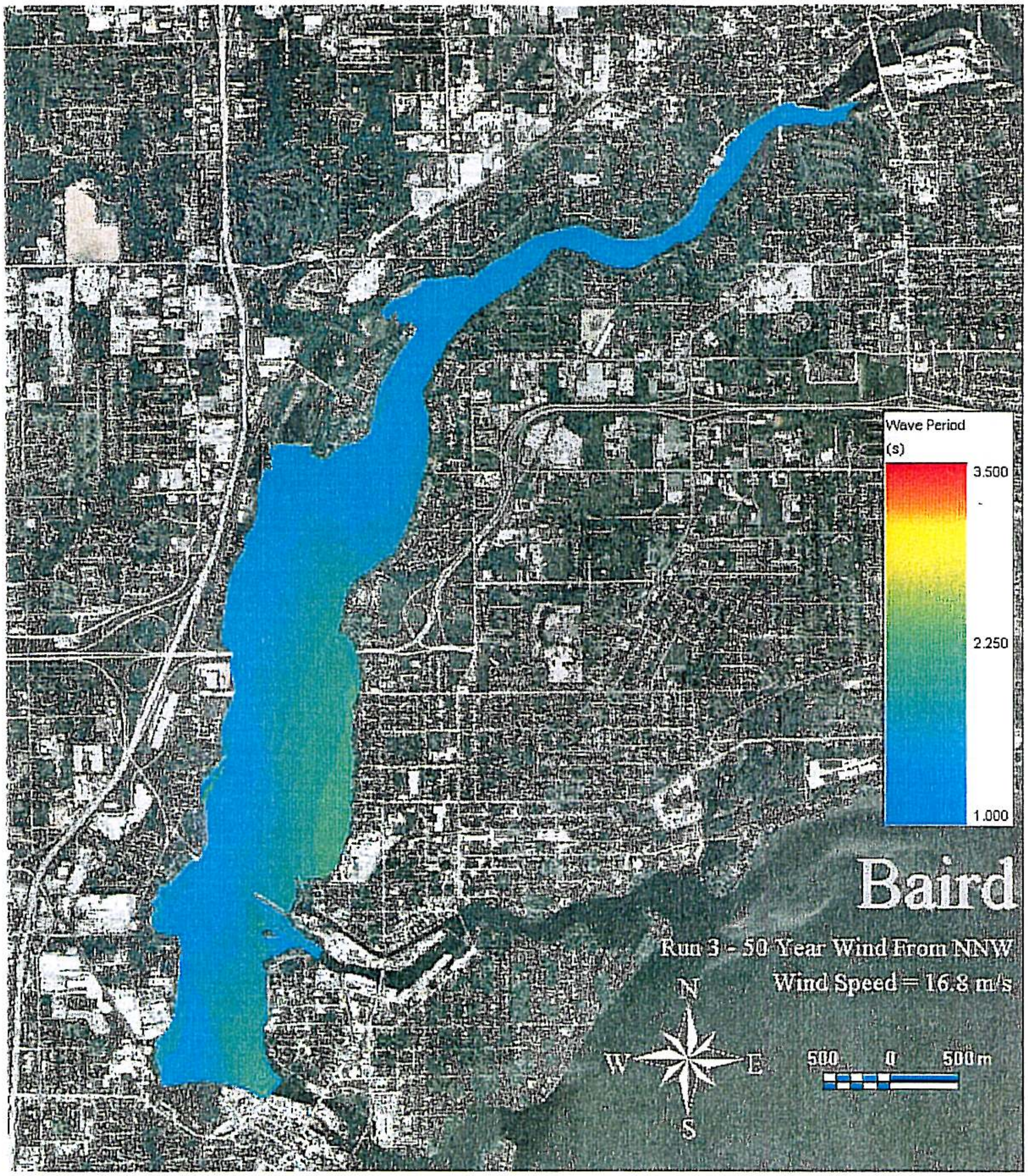
STWAVE RESULTS

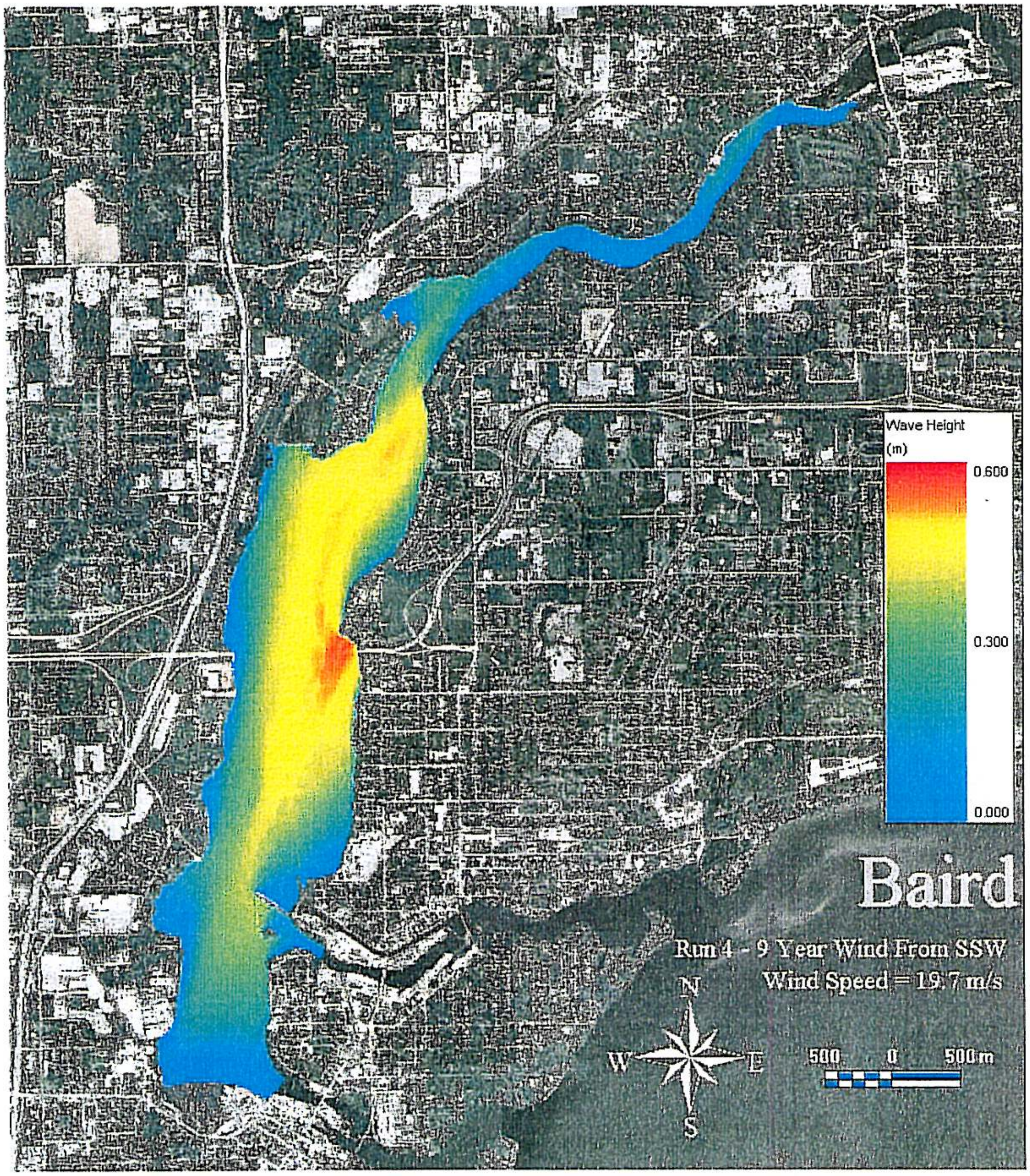
FOR RUNS 2 TO 5 ON OU1, FOX RIVER

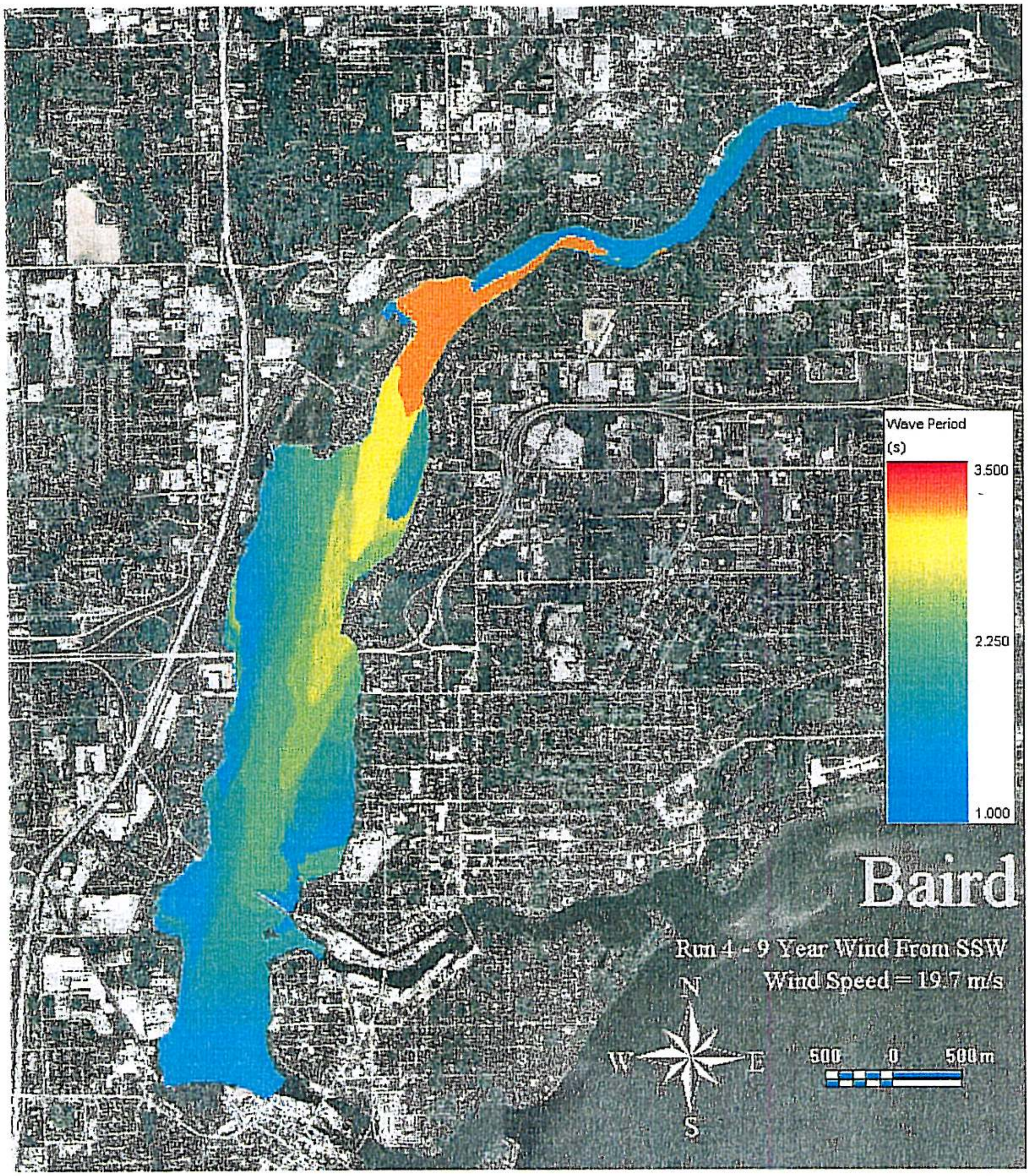

Wave Height
(m)
0.600
0.300
0.000
Baird
Run 2 - 50 Year Wind From WSW
Wind Speed = 22.6 m/s
N
W
E
S
500
0
500 m

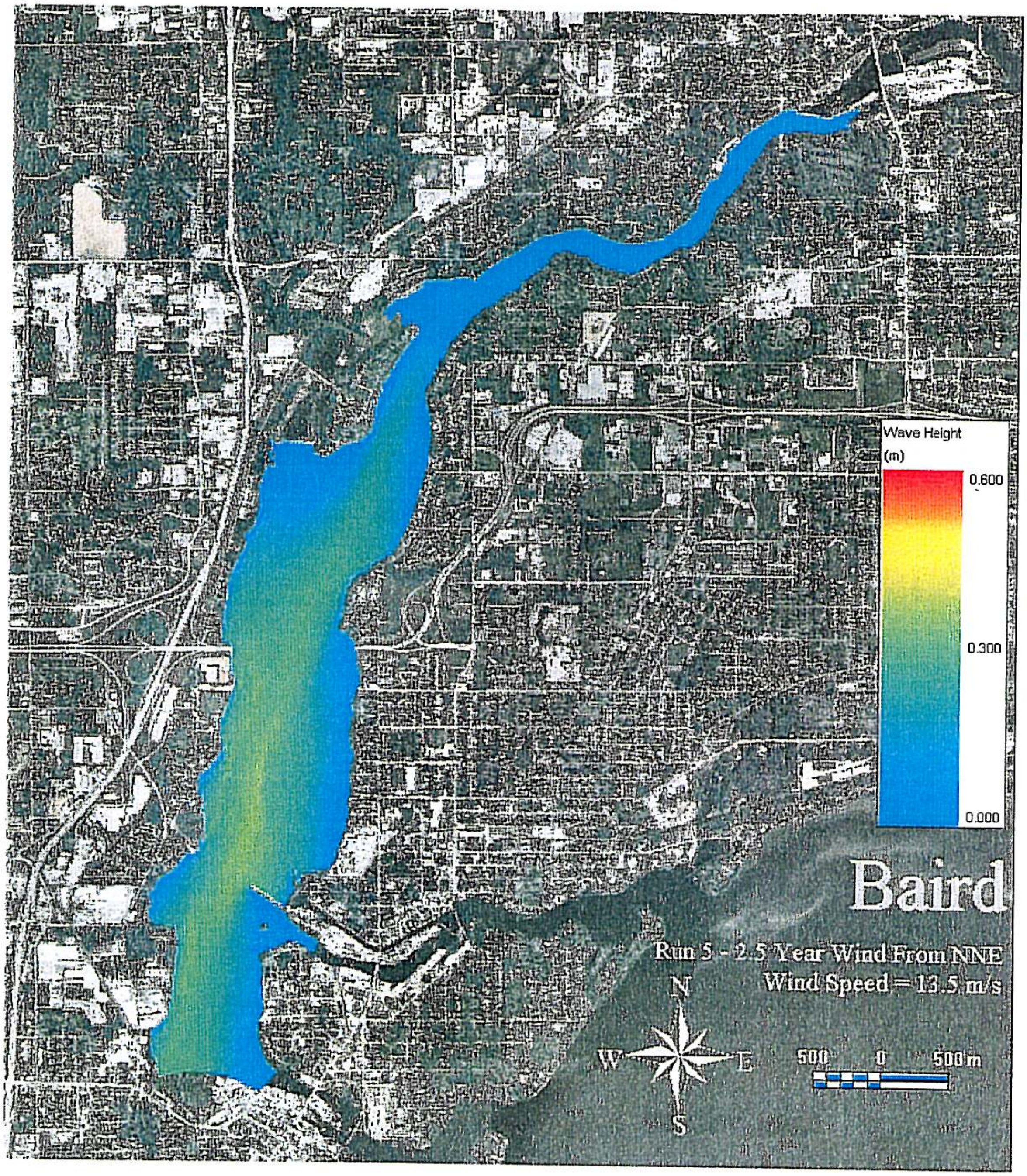

Wave Period
(s)
3.500
2.250
1.000
Baird
Run 2 - 50 Year Wind From WSW
Wind Speed = 22.6 m/s
N
W
E
S
500
0
500 m


Wave Height
(m)
0.600
0.300
0.000
Baird
Run 3 - 50 Year Wind From NNW
Wind Speed = 16.8 m/s
N
W
E
S
500 0 500m


Wave Period
(s)
3.500
2.250
1.000
Baird
Run 3 - 50 Year Wind From NNW
Wind Speed = 16.8 m/s
N
W
E
S
500
0
500 m


Wave Height
(m)
0.600
0.300
0.000
Baird
Run 4 - 9 Year Wind From SSW
Wind Speed = 19.7 m/s
N
W
E
S
500
0
500 m


Wave Period
(s)
3.500
2.250
1.000
Baird
Run 4 - 9 Year Wind From SSW
Wind Speed = 19.7 m/s
N
W
E
S
500
0
500 m

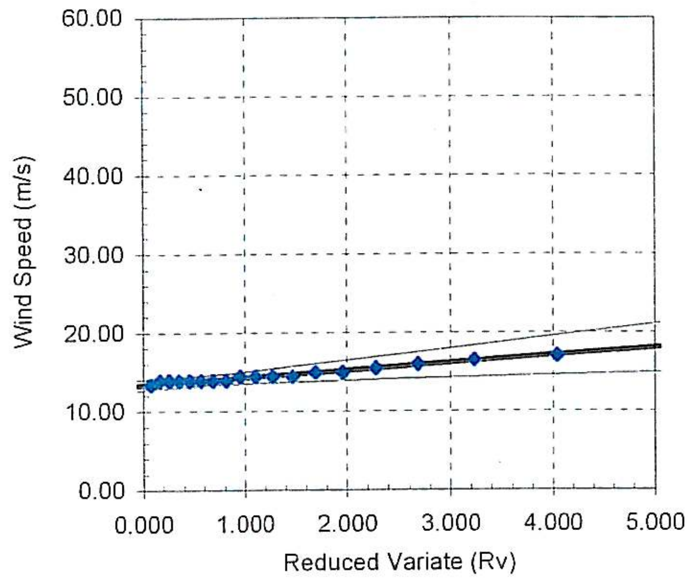

Wave Height
(m)
0.600
0.300
0.000
Baird
Run 5 - 2.5 Year Wind From NNE
Wind Speed = 13.5 m/s
N
W
E
S
500
0
500 m

APPENDIX B

PEAK OVER THRESHOLD ANALYSIS FOR EACH DIRECTION AND SEASON

Peak over Threshold Extreme Value Analysis

Data Set: Green Bay Winds 1978 - 2002 Full year: 180-225deg only



Fisher-Tippett II

Total Years of Data:	24
Total Storm Events:	32
Total No. Events Selected:	32
Events per year:	1.33

Sample Statistics

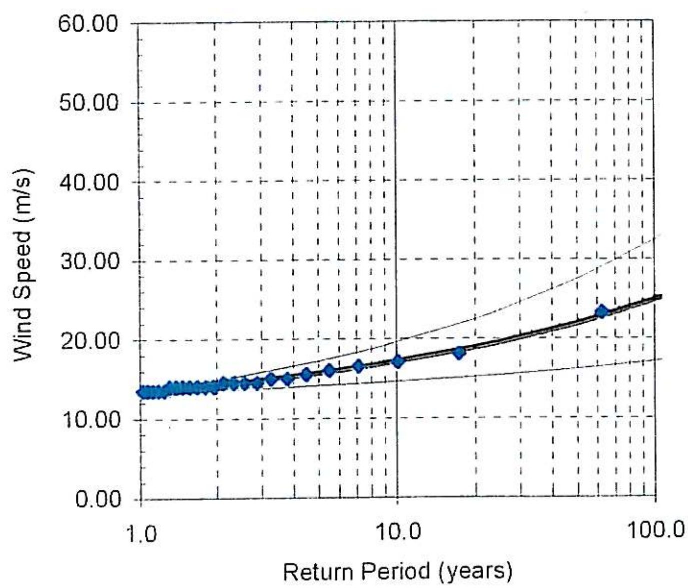
Mean:	14.50
Maximum:	23.14
Minimum:	13.36
s:	1.96
Sample skewness:	-0.34

FTII Parameters

Shape:	3.00
Scale:	0.928
Location:	13.374

Goodness of Fit

Correlation:	0.995



Return Period

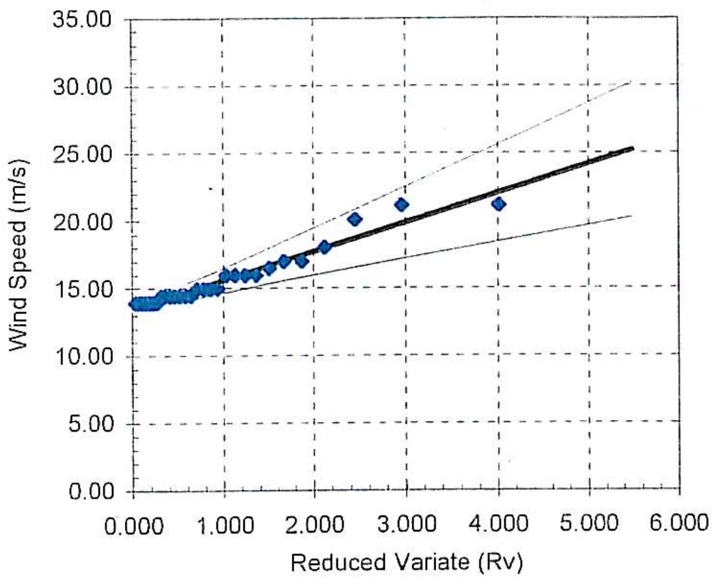
		Confidence Limit	
Tr	X(T)	Upper	Lower
1	13.09	13.8	12.4
2	14.17	14.9	13.5
5	15.69	17.2	14.2
8.2	16.67	18.8	14.5
20	18.85	22.5	15.2
25	19.50	23.6	15.4
50	21.85	27.5	16.2
100	24.79	32.4	17.1
200	28.49	38.7	18.3
500	34.90	49.4	20.4

Date: 15-Jun-07

Baird

Peak over Threshold Extreme Value Analysis

Data Set: Green Bay Winds 1978 - 2002 Full Year: 225-280



Three-Parameter Weibull Distribution

Total Years of Data:	24
Total Storm Events:	29
Total No. Events Selected:	29
Events per year:	1.21

Sample Statistics

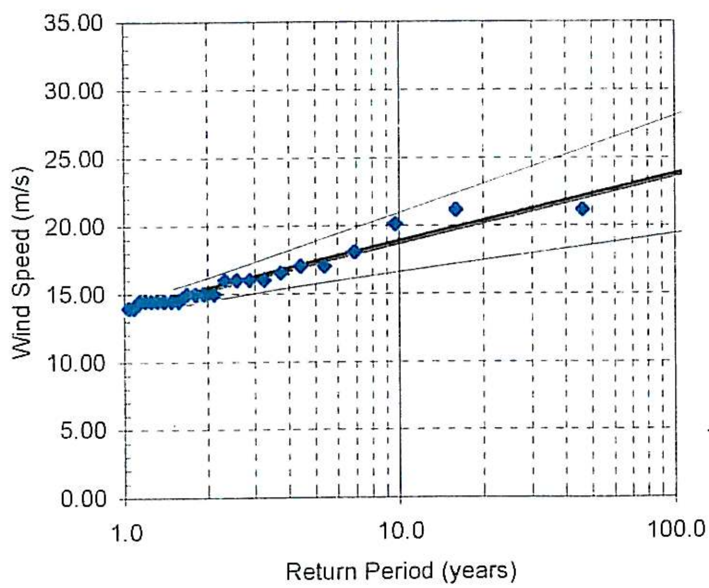
Mean:	15.58
Maximum:	21.08
Minimum:	13.88
s:	2.10
Sample skewness:	-1.04

Weibull Parameters

Shape:	1.00
Scale:	2.149
Location:	13.435

Goodness of Fit

Correlation:	0.974



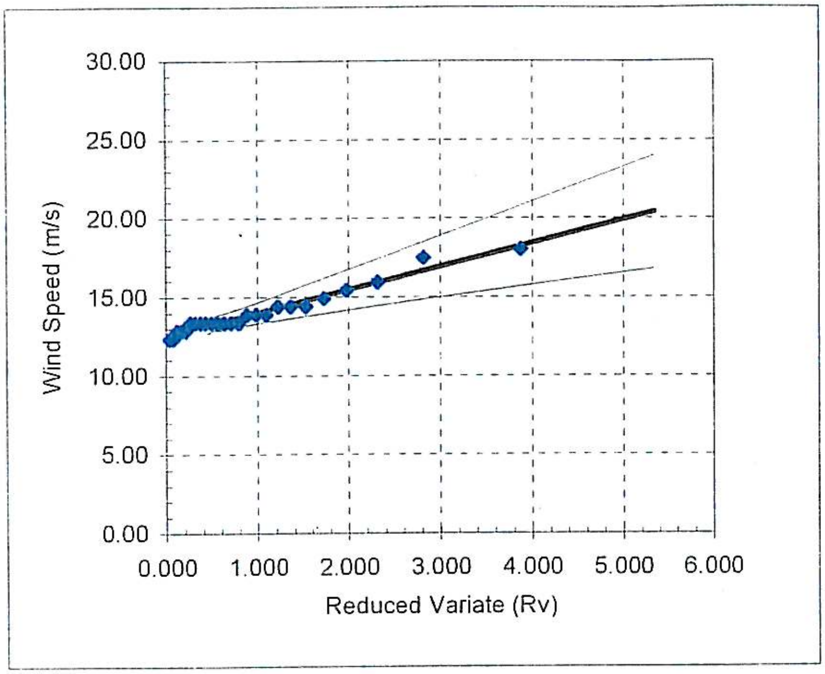
Return Period

		Confidence Limit	
Tr	X(T)	Upper	Lower
1.5	14.71	15.4	14.0
2	15.33	16.2	14.5
3.73	16.67	18.0	15.4
10	18.79	21.0	16.6
15	19.66	22.2	17.1
20	20.28	23.1	17.5
25	20.76	23.8	17.7
50	22.25	25.9	18.6
100	23.74	28.1	19.4
200	25.23	30.2	20.2

Baird

Peak over Threshold Extreme Value Analysis

Data Set: Green Bay Winds 1978 - 2002 Full Year: 340-20 Deg.



Three-Parameter Weibull Distribution

Total Years of Data:	24
Total Storm Events:	25
Total No. Events Selected:	25
Events per year:	1.04

Sample Statistics

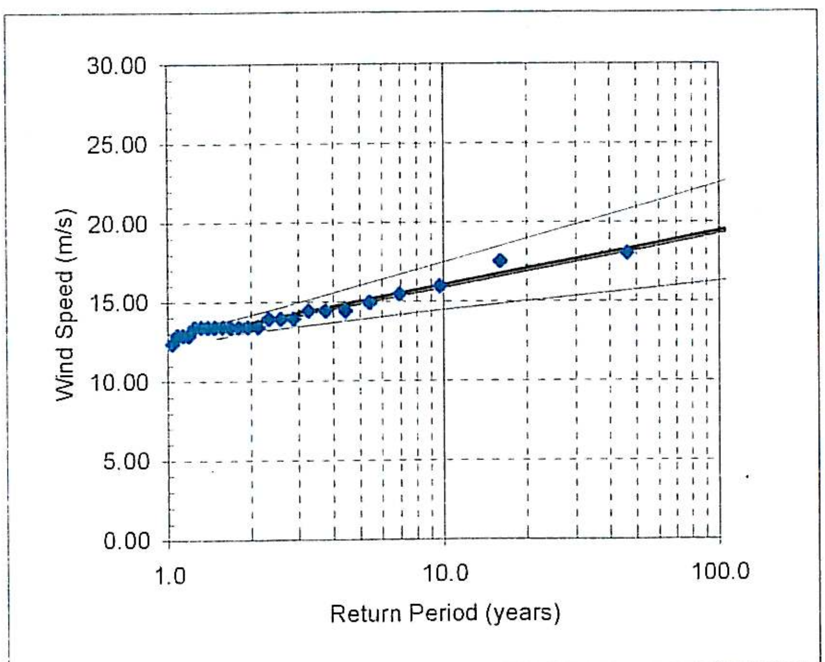
Mean:	14.00
Maximum:	17.99
Minimum:	12.33
s:	1.42
Sample skewness:	-0.56

Weibull Parameters

Shape:	1.00
Scale:	1.471
Location:	12.532

Goodness of Fit

Correlation:	0.981



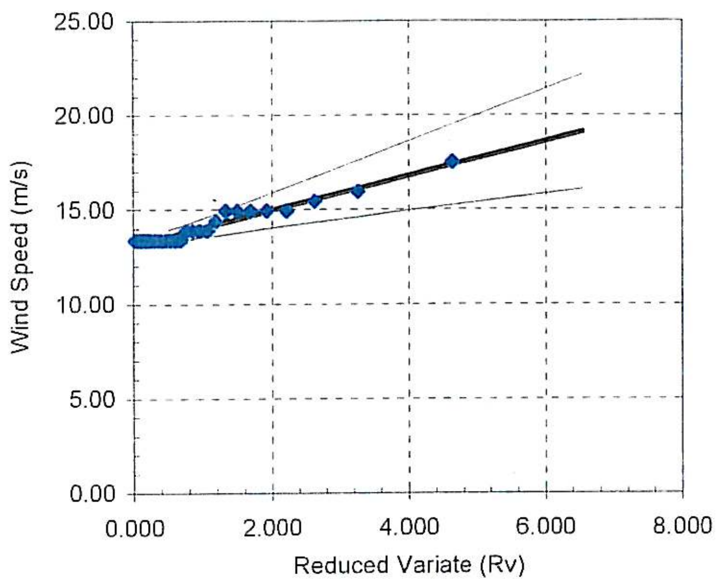
Return Period

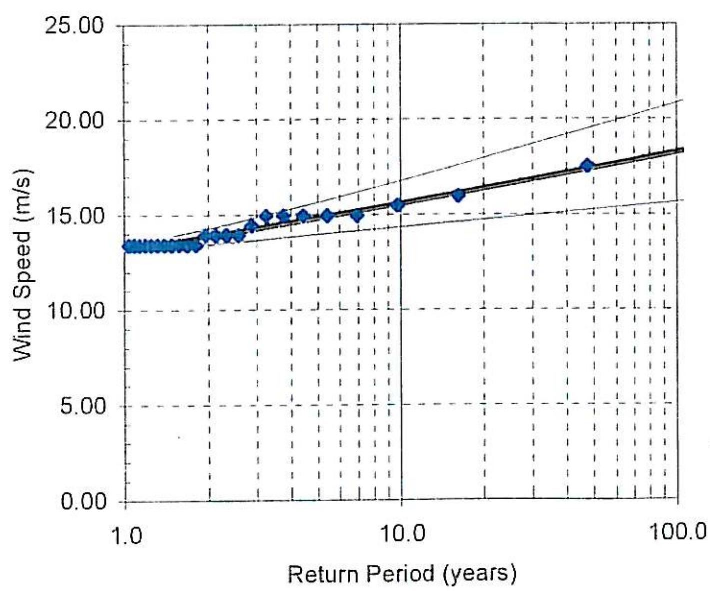
		Confidence Limit	
Tr	X(T)	Upper	Lower
1.5	13.19	13.7	12.7
2	13.61	14.2	13.1
2.5	13.94	14.6	13.3
10	15.98	17.5	14.5
15	16.58	18.4	14.8
20	17.00	19.0	15.0
25	17.33	19.5	15.2
50	18.35	21.0	15.7
100	19.37	22.5	16.3
200	20.39	24.0	16.8

Baird

Peak over Threshold Extreme Value Analysis

Data Set: Green Bay Winds 1978 - 2002 Full Year: 20-60 Deg.





Three-Parameter Weibull Distribution

Total Years of Data:	24
Total Storm Events:	27
Total No. Events Selected:	27
Events per year:	1.13

Sample Statistics

Mean:	14.08
Maximum:	17.48
Minimum:	13.36
s:	1.03
Sample skewness:	-0.11

Weibull Parameters

Shape:	0.90
Scale:	0.915
Location:	13.125

Goodness of Fit

Correlation:	0.977

Return Period

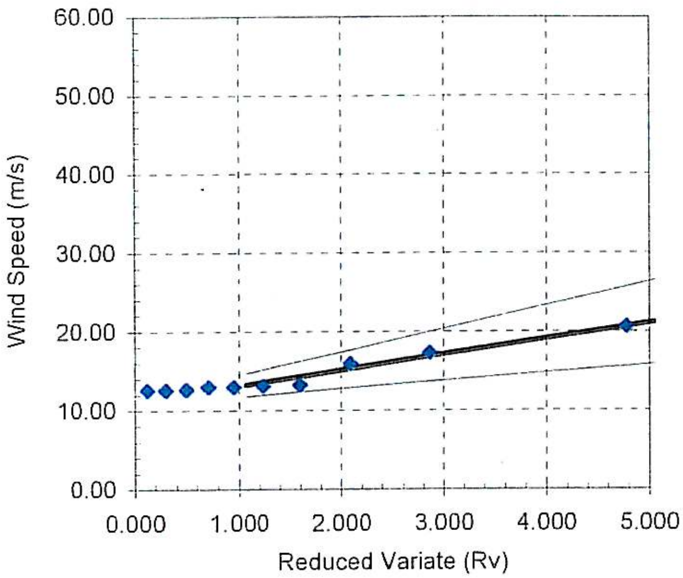
Tr	X(T)	Confidence Limit Upper	Lower
1.5	13.57	13.9	13.2
2	13.85	14.3	13.4
5	14.80	15.6	14.0
10	15.57	16.8	14.3
15	16.03	17.5	14.6
20	16.36	18.0	14.7
26.2	16.67	18.5	14.9
50	17.43	19.6	15.3
100	18.26	20.9	15.7
200	19.10	22.1	16.1

Date: 15-Jun-07

Baird

Peak over Threshold Extreme Value Analysis

Data Set: Appleton Winds 1998 - 2004 Full Year: 180-225deg only



Fisher-Tippet II

Total Years of Data:	5
Total Storm Events:	16
Total No. Events Selected:	16
Events per year:	3.20

Sample Statistics

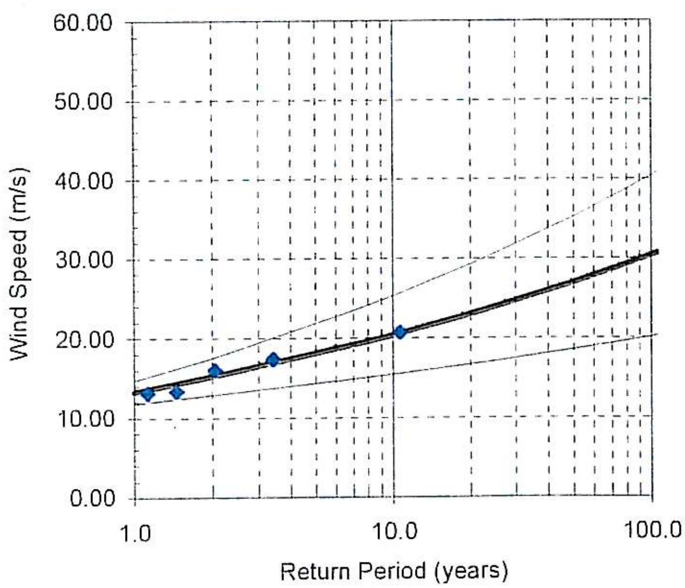
Mean:	13.28
Maximum:	20.564
Minimum:	11.176
s:	2.54
Sample skewness:	-2.84

FTII Parameters

Shape:	6.00
Scale:	1.976
Location:	11.169

Goodness of Fit

Correlation:	0.978



Return Period

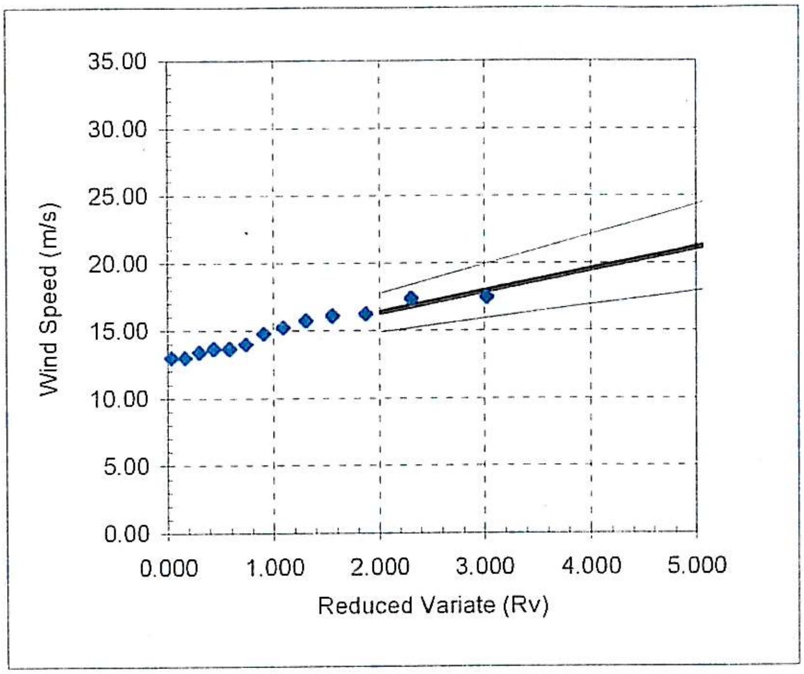
		Confidence Limit	
Tr	X(T)	Upper	Lower
1	13.28	14.7	11.8
2	15.24	17.6	12.9
8.2	19.69	24.2	15.1
10	20.38	25.3	15.5
20	23.00	29.3	16.7
25	23.90	30.7	17.1
50	26.92	35.3	18.6
100	30.31	40.4	20.2
200	34.12	46.3	22.0
500	39.86	55.0	24.7

Date: 15-Jun-07

Baird

Peak over Threshold Extreme Value Analysis

Data Set: Appleton Winds 1998 - 2002 Full Year: 225-280



Fisher-Tippet I (Gumbel)

Total Years of Data:	5
Total Storm Events:	20
Total No. Events Selected:	20
Events per year:	4.00

Sample Statistics

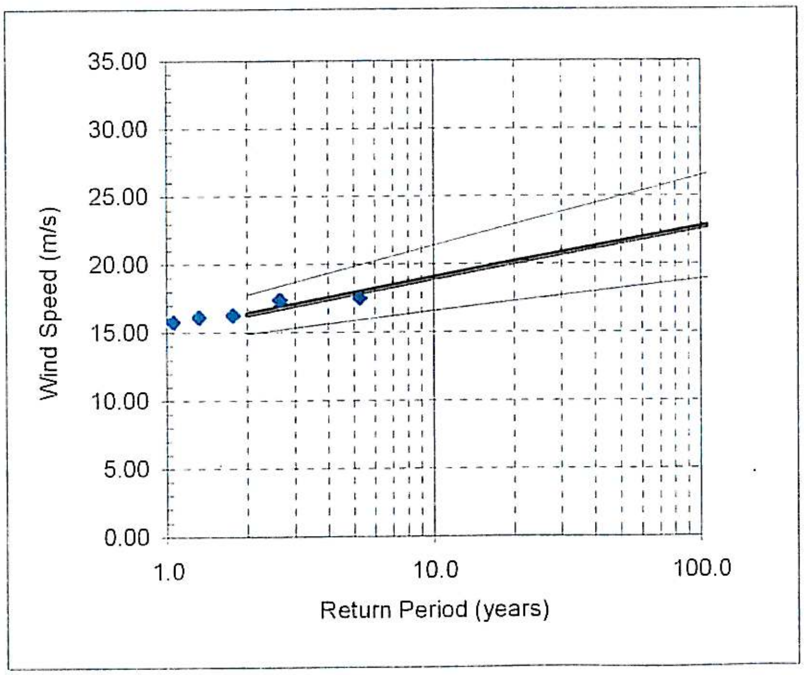
Mean:	13.95
Maximum:	17.435
Minimum:	12.07
s:	1.79
Sample skewness:	-1.13

FT I Parameters

Scale:	1.600
Location:	13.113

Goodness of Fit

Correlation:	0.974



Return Period

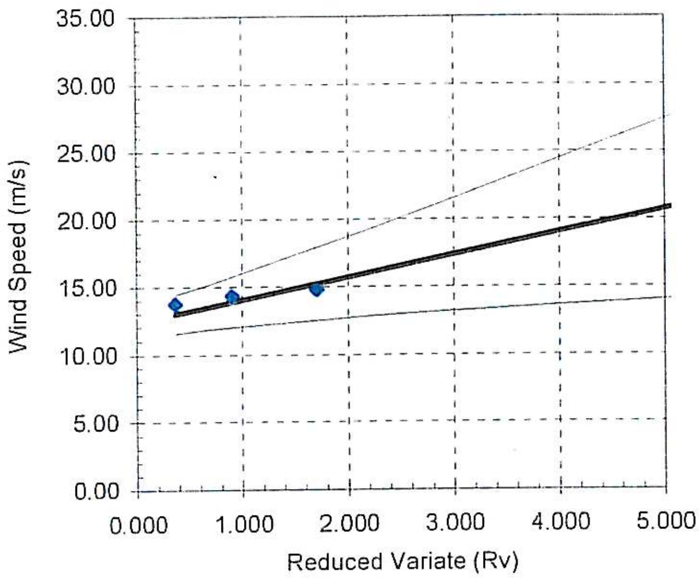
		Confidence Limit	
Tr	X(T)	Upper	Lower
2	16.33	17.78	14.89
3.7	17.37	19.18	15.56
10	19.00	21.40	16.59
20	20.11	22.94	17.29
25	20.47	23.44	17.51
50	21.59	24.97	18.20
100	22.70	26.50	18.89
200	23.81	28.04	19.58
500	25.28	30.06	20.49
1000	26.38	31.60	21.17

Date: 15-Jun-07

Baird

Peak over Threshold Extreme Value Analysis

Data Set: Appleton Winds 1998 - 2004 Full Year: 340-20deg only



Fisher-Tippet I (Gumbel)

Total Years of Data:	5
Total Storm Events:	5
Total No. Events Selected:	5
Events per year:	1.00

Sample Statistics

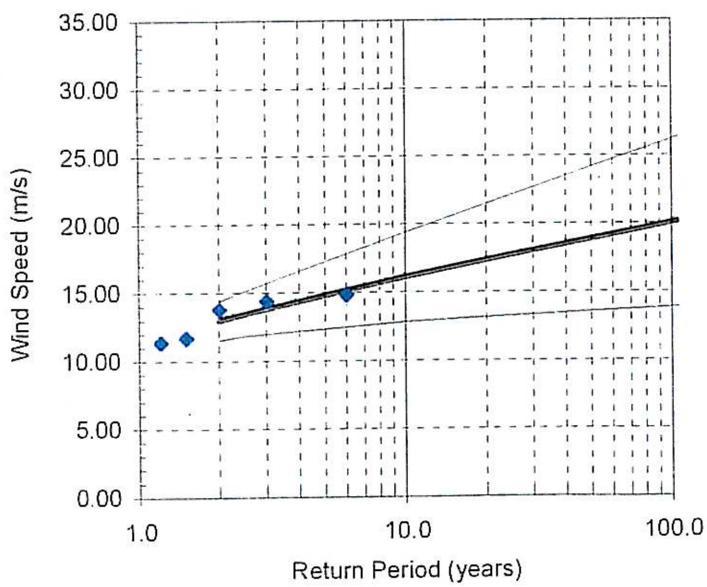
Mean:	13.14
Maximum:	14.752
Minimum:	11.288
s:	1.58
Sample skewness:	-4.10

FT I Parameters

Scale:	1.666
Location:	12.375

Goodness of Fit

Correlation:	0.932



Return Period

		Confidence Limit	
Tr	X(T)	Upper	Lower
2	12.99	14.43	11.54
2.5	13.49	15.14	11.85
10	16.12	19.41	12.84
20	17.32	21.47	13.18
25	17.70	22.13	13.28
50	18.88	24.17	13.58
100	20.04	26.21	13.87
200	21.20	28.24	14.15
500	22.73	30.93	14.52
1000	23.88	32.97	14.80

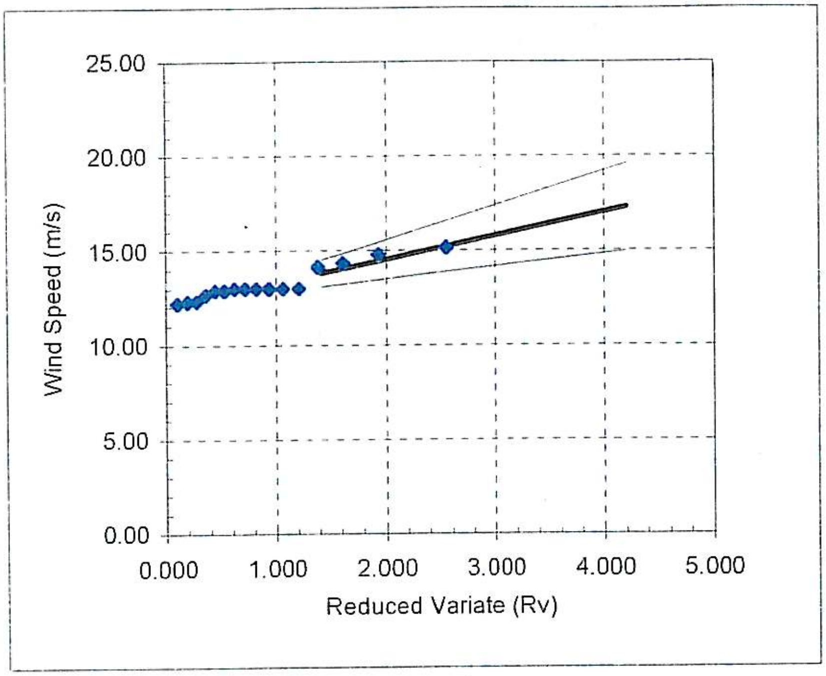
Date: 15-Jun-07

Baird

Peak over Threshold Extreme Value Analysis

Data Set: Appleton Winds 1978 - 2002 Full Year:20-60

Three-Parameter Weibull Distribution



Total Years of Data:	5
Total Storm Events:	16
Total No. Events Selected:	16
Events per year:	3.20

Sample Statistics

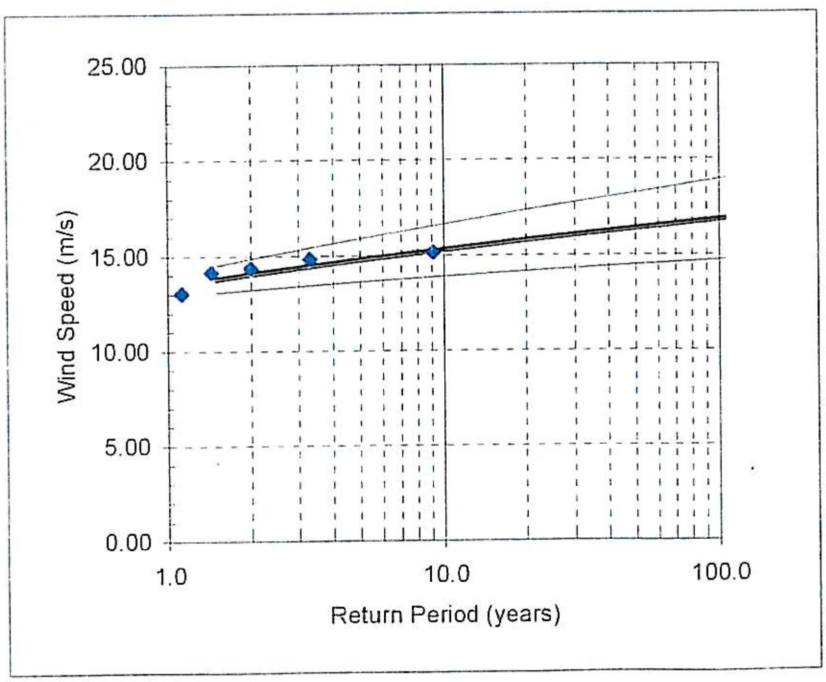
Mean:	13.19
Maximum:	15.09
Minimum:	12.18
s:	0.88
Sample skewness:	-0.17

Weibull Parameters

Shape:	1.30
Scale:	1.241
Location:	12.049

Goodness of Fit

Correlation:	0.957



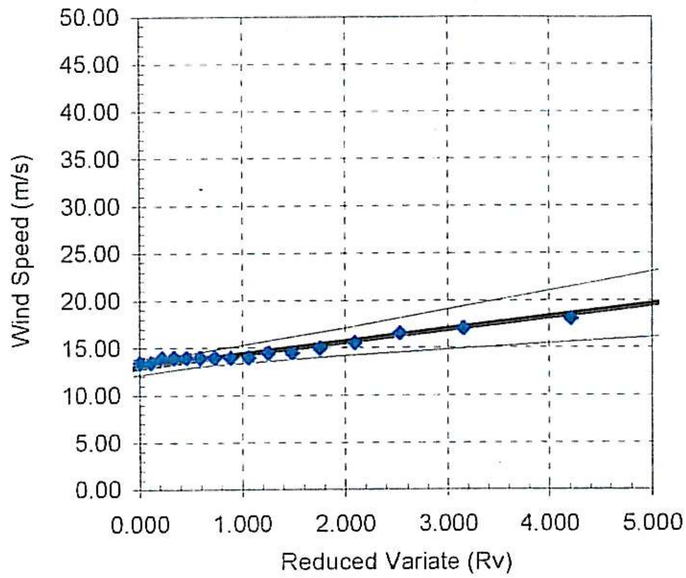
Return Period

Tr	X(T)	Confidence Limit Upper	Confidence Limit Lower
1.5	13.80	14.5	13.1
2	14.05	14.9	13.2
5	14.77	15.9	13.6
10	15.28	16.7	13.9
15	15.57	17.1	14.1
20	15.76	17.4	14.2
25	15.92	17.6	14.2
50	16.38	18.3	14.5
78	16.67	18.7	14.6
200	17.26	19.6	14.9

Baird

Peak over Threshold Extreme Value Analysis

Data Set: Green Bay Winds 1978 - 2002 Ice Removed: 180-225deg only **Fisher-Tippet II**



Total Years of Data:	24
Total Storm Events:	26
Total No. Events Selected:	26
Events per year:	1.08

Sample Statistics

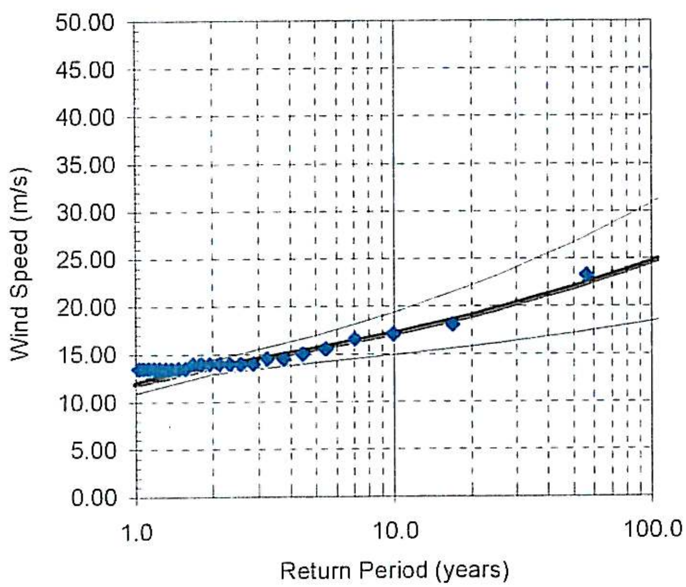
Mean:	14.53
Maximum:	23.14
Minimum:	13.36
s:	2.14
Sample skewness:	-0.51

FTII Parameters

Shape:	4.00
Scale:	1.307
Location:	13.020

Goodness of Fit

Correlation:	0.985



Return Period

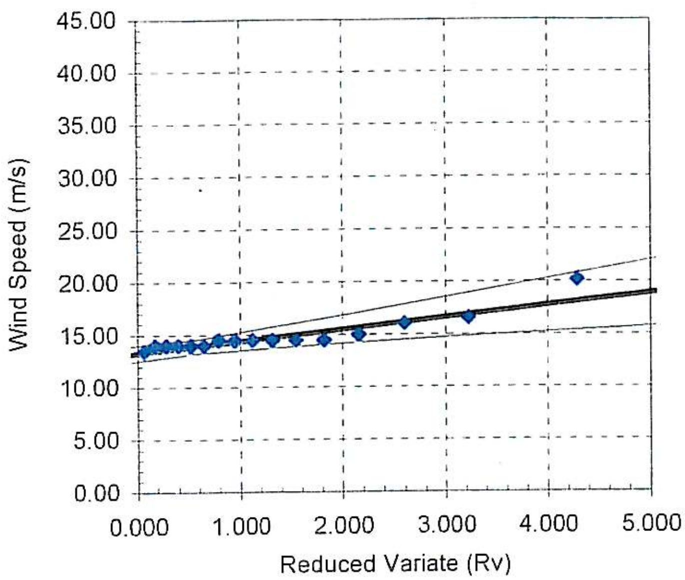
Tr	X(T)	Confidence Limit Upper	Lower
1	11.92	12.9	10.9
2	13.69	14.4	13.0
5	15.57	17.0	14.1
10	17.16	19.4	14.9
20	19.01	22.2	15.8
25	19.66	23.2	16.1
50	21.94	26.7	17.2
100	24.64	30.8	18.5
200	27.84	35.7	20.0
500	33.01	43.7	22.4

Date: 15-Jun-07

Baird

Peak over Threshold Extreme Value Analysis

Data Set: Green Bay Winds 1978 - 2002 Ice removed: 225-280



Fisher-Tippet II

Total Years of Data:	24
Total Storm Events:	27
Total No. Events Selected:	27
Events per year:	1.13

Sample Statistics

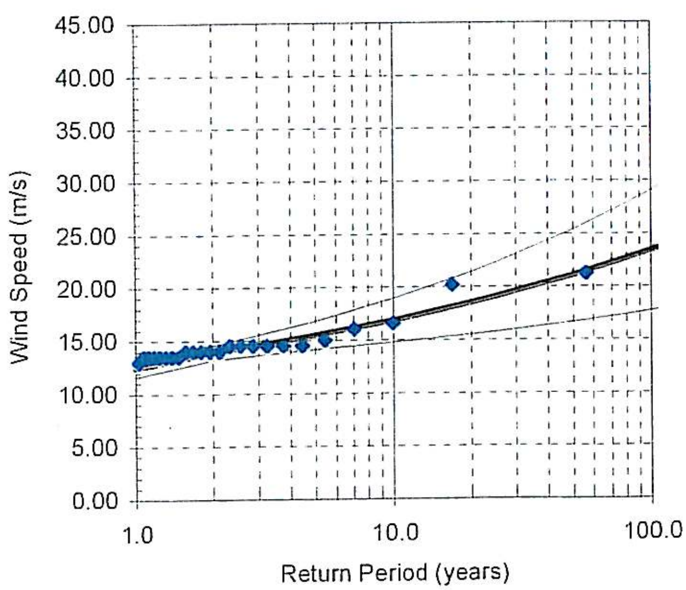
Mean:	14.41
Maximum:	21.08
Minimum:	12.85
s:	1.98
Sample skewness:	-0.85

FTII Parameters

Shape:	4.00
Scale:	1.124
Location:	13.246

Goodness of Fit

Correlation:	0.959



Return Period

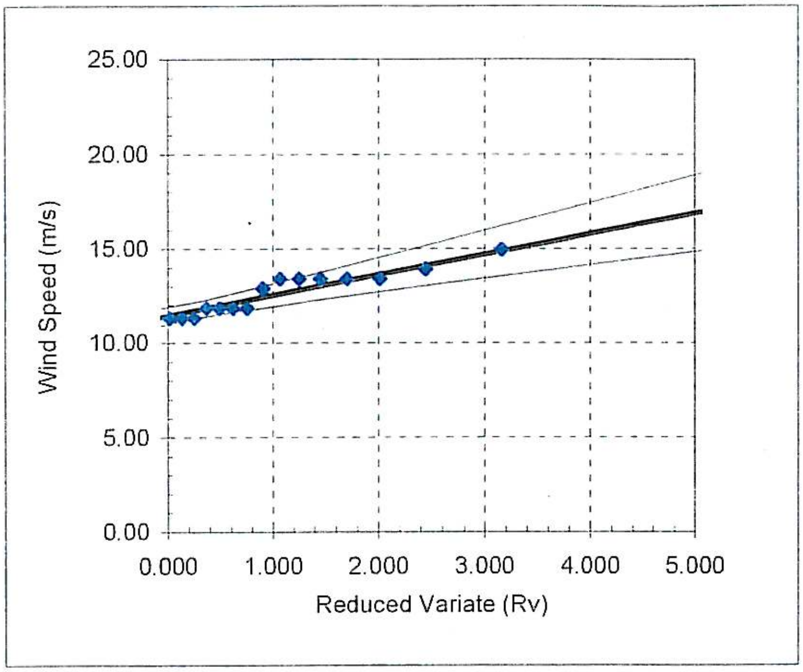
		Confidence Limit	
Tr	X(T)	Upper	Lower
1	12.44	13.3	11.6
2	13.88	14.6	13.2
5	15.51	16.8	14.2
10	16.89	18.9	14.8
20	18.49	21.4	15.5
25	19.06	22.3	15.8
50	21.03	25.4	16.7
100	23.37	29.0	17.7
200	26.15	33.4	18.9
500	30.64	40.4	20.9

Date: 15-Jun-07

Baird

Peak over Threshold Extreme Value Analysis

Data Set: Green Bay Winds 1978 - 2002 Ice Removed: 340-20 Deg.



Fisher-Tippet I (Gumbel)

Total Years of Data:	24
Total Storm Events:	23
Total No. Events Selected:	23
Events per year:	0.96

Sample Statistics

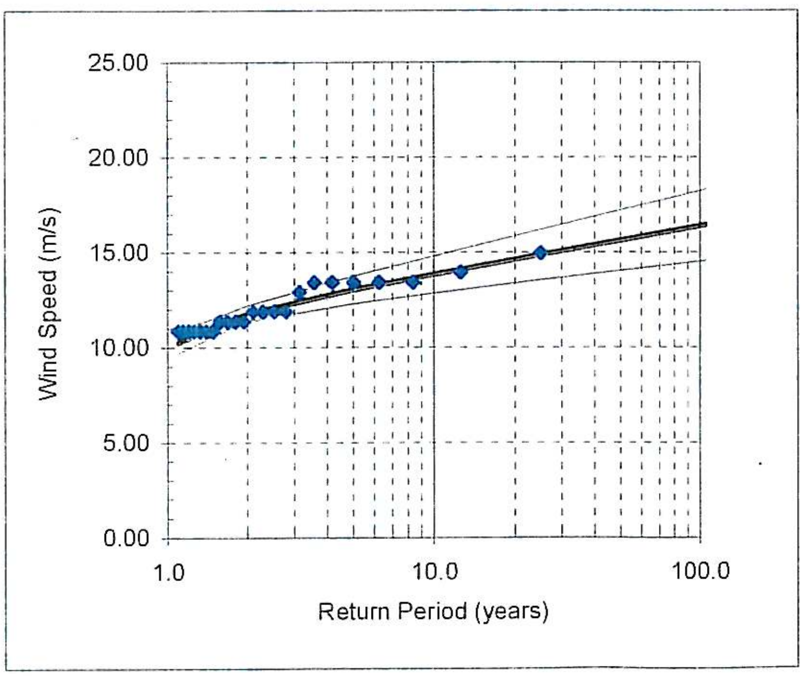
Mean:	12.02
Maximum:	14.9
Minimum:	10.79
s:	1.25
Sample skewness:	-0.21

FT I Parameters

Scale:	1.083
Location:	11.447

Goodness of Fit

Correlation:	0.960



Return Period

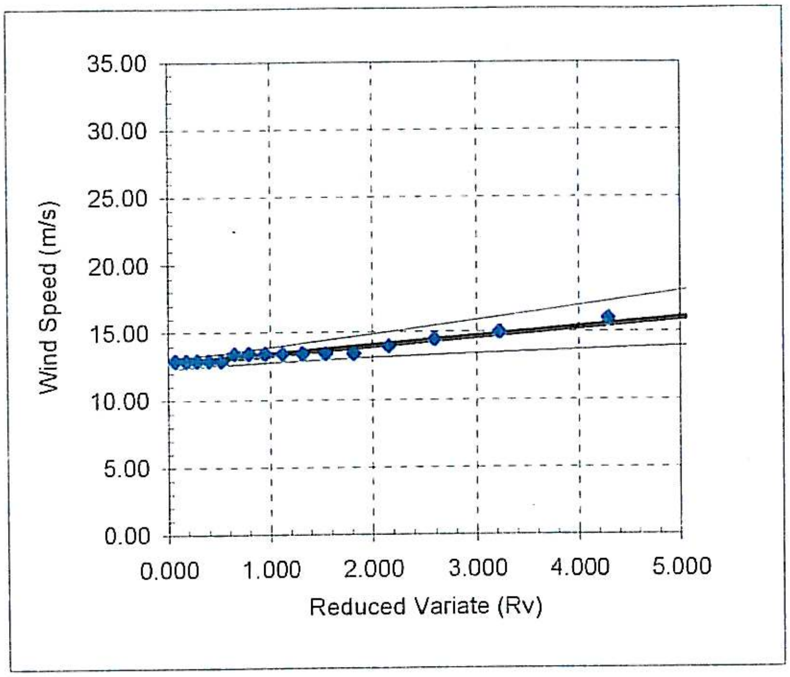
		Confidence Limit	
Tr	X(T)	Upper	Lower
1.1	10.27	10.83	9.71
2	11.78	12.21	11.35
5	13.02	13.74	12.30
10	13.84	14.82	12.86
20	14.62	15.86	13.38
25	14.86	16.19	13.54
50	15.63	17.22	14.04
100	16.38	18.24	14.53
200	17.14	19.26	15.02
500	18.13	20.60	15.66

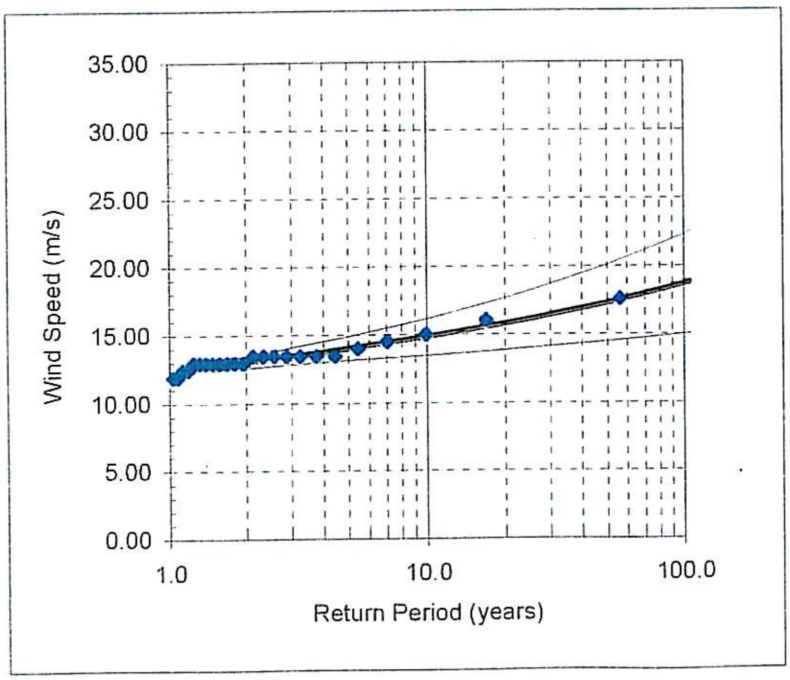
Date: 15-Jun-07

Baird

Peak over Threshold Extreme Value Analysis

Data Set: Green Bay Winds 1978 - 2002 Ice Removed: 20-60 Deg.





Fisher-Tippet II

Total Years of Data:	24
Total Storm Events:	27
Total No. Events Selected:	27
Events per year:	1.13

Sample Statistics

Mean:	13.21
Maximum:	17.48
Minimum:	11.82
s:	1.28
Sample skewness:	-0.55

FTII Parameters

Shape:	4.00
Scale:	0.662
Location:	12.677

Goodness of Fit

Correlation:	0.983

Return Period

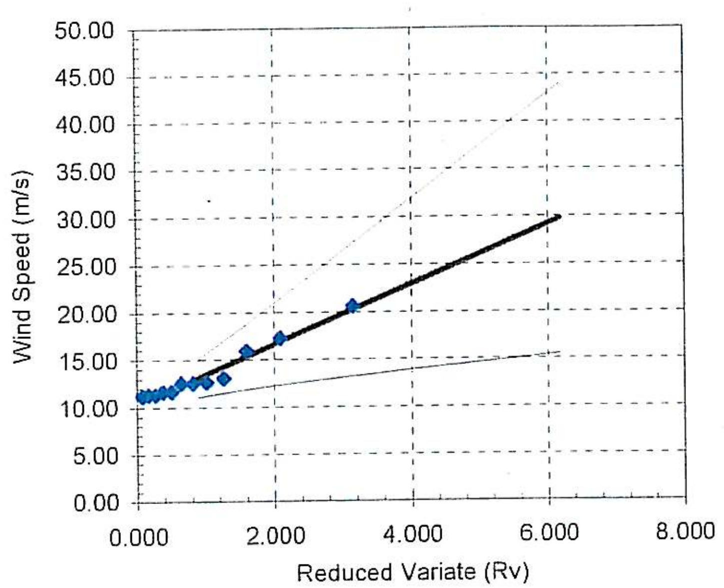
		Confidence Limit	
Tr	X(T)	Upper	Lower
1.5	12.75	13.2	12.3
2	13.05	13.5	12.6
5	14.01	14.9	13.1
10	14.82	16.2	13.5
20	15.76	17.7	13.9
25	16.10	18.2	14.0
50	17.27	20.1	14.4
100	18.65	22.3	15.0
200	20.28	25.0	15.6
500	22.93	29.3	16.6

Date: 15-Jun-07

Baird

Peak over Threshold Extreme Value Analysis

Data Set: Appleton Winds 1998 - 2004 Ice Removed: 180-225deg only **Three-Parameter Weibull Distribution**



Total Years of Data:	5
Total Storm Events:	12
Total No. Events Selected:	12
Events per year:	2.40

Sample Statistics

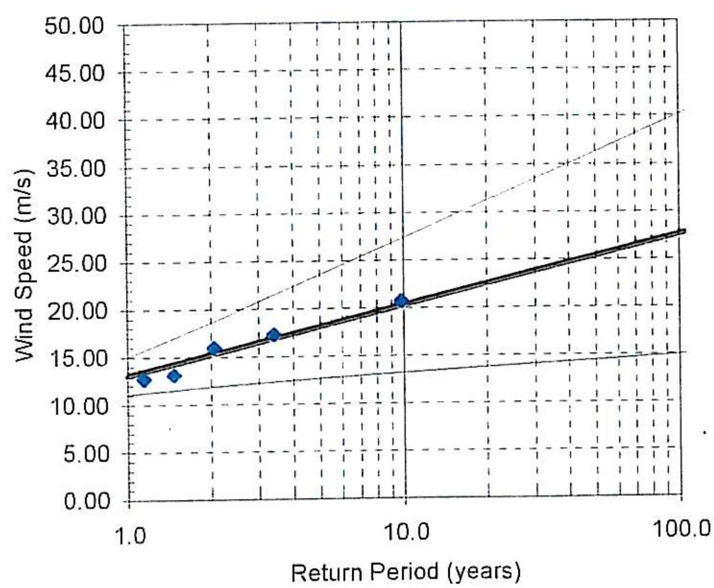
Mean:	13.44
Maximum:	20.56
Minimum:	11.18
s:	2.93
Sample skewness:	-5.01

Weibull Parameters

Shape:	1.00
Scale:	3.149
Location:	10.308

Goodness of Fit

Correlation:	0.980



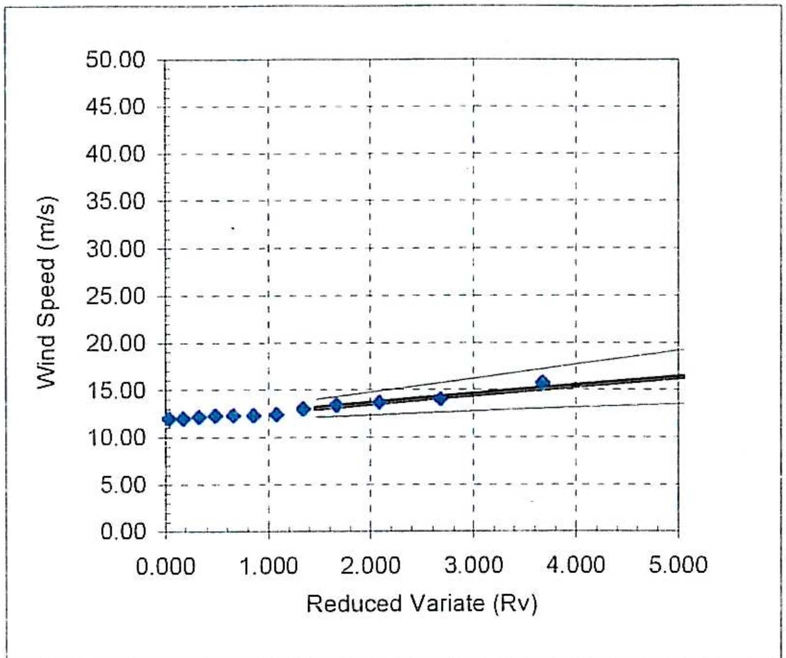
Return Period

		Confidence Limit	
Tr	X(T)	Upper	Lower
1	13.06	15.0	11.1
2	15.25	18.6	11.9
5	18.13	23.6	12.7
10	20.31	27.4	13.2
15	21.59	29.6	13.5
20	22.50	31.2	13.8
25	23.20	32.5	13.9
50	25.38	36.3	14.5
100	27.56	40.2	15.0
200	29.75	44.0	15.5

Baird

Peak over Threshold Extreme Value Analysis

Data Set: Appleton Winds 1978 - 2002 Ice removed: 225-280



Fisher-Tippet II

Total Years of Data:	5
Total Storm Events:	20
Total No. Events Selected:	20
Events per year:	4.00

Sample Statistics

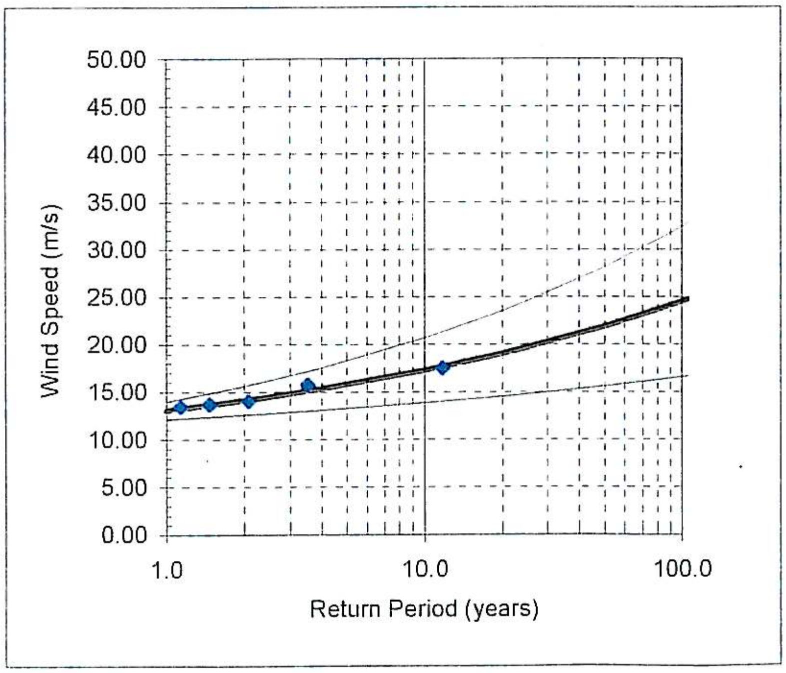
Mean:	12.70
Maximum:	17.435
Minimum:	11.4
s:	1.53
Sample skewness:	-0.51

FTII Parameters

Shape:	4.00
Scale:	0.919
Location:	11.728

Goodness of Fit

Correlation:	0.989



Return Period

Tr	X(T)	Confidence Limit Upper	Confidence Limit Lower
1	13.07	14.0	12.1
2	14.13	15.7	12.6
3.7	15.20	17.4	13.1
10	17.27	20.7	13.9
20	19.03	23.5	14.5
25	19.67	24.6	14.8
50	21.87	28.1	15.6
100	24.49	32.4	16.6
200	27.61	37.4	17.8
500	32.64	45.6	19.7

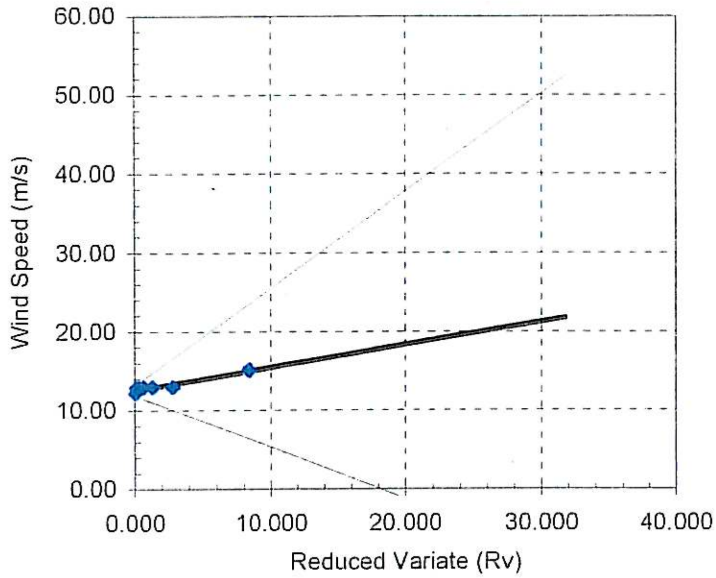
Date: 15-Jun-07

Baird

Peak over Threshold Extreme Value Analysis

Data Set: Appleton Winds 1978 - 2002 Ice removed: 20-60

Three-Parameter Weibull Distribution



Total Years of Data:	5
Total Storm Events:	7
Total No. Events Selected:	7
Events per year:	1.40

Sample Statistics

Mean:	13.12
Maximum:	15.09
Minimum:	12.18
s:	0.91
Sample skewness:	-0.21

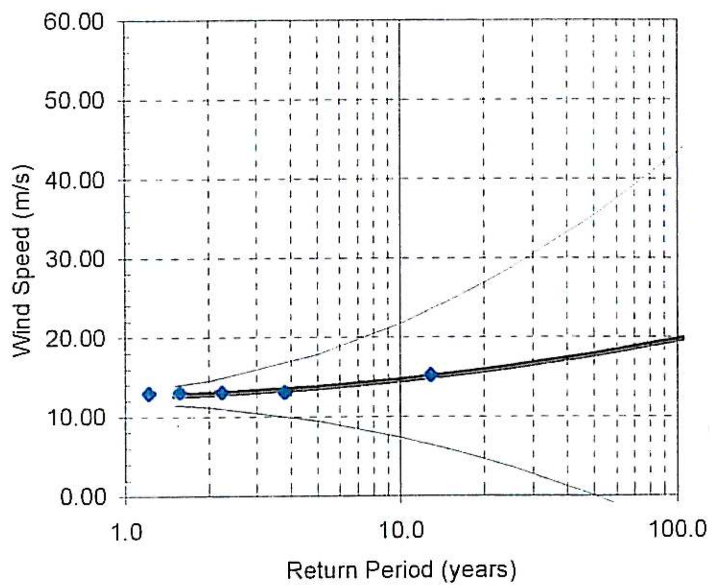
Weibull Parameters

Shape:	0.50
Scale:	0.289
Location:	12.568

Goodness of Fit

Correlation:	0.950

Error - k out of range



Return Period

		Confidence Limit	
Tr	X(T)	Upper	Lower
1.5	12.73	14.0	11.5
2	12.87	14.6	11.2
5	13.66	17.9	9.5
10	14.58	21.8	7.4
15	15.24	24.6	5.9
20	15.77	26.9	4.7
25	16.22	28.8	3.7
50	17.78	35.5	0.1
100	19.62	43.3	-4.1
200	21.74	52.4	-8.9

Baird